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(FOURTH SERIES).

EDITED BY MAJOR W. A. HARRISON,
SECRETARY, R.E. INSTITUTE.

PAPER 2.

NOTES ON ROOFS.

BY

MAJOR E. N. STOCKLEY, R.E.

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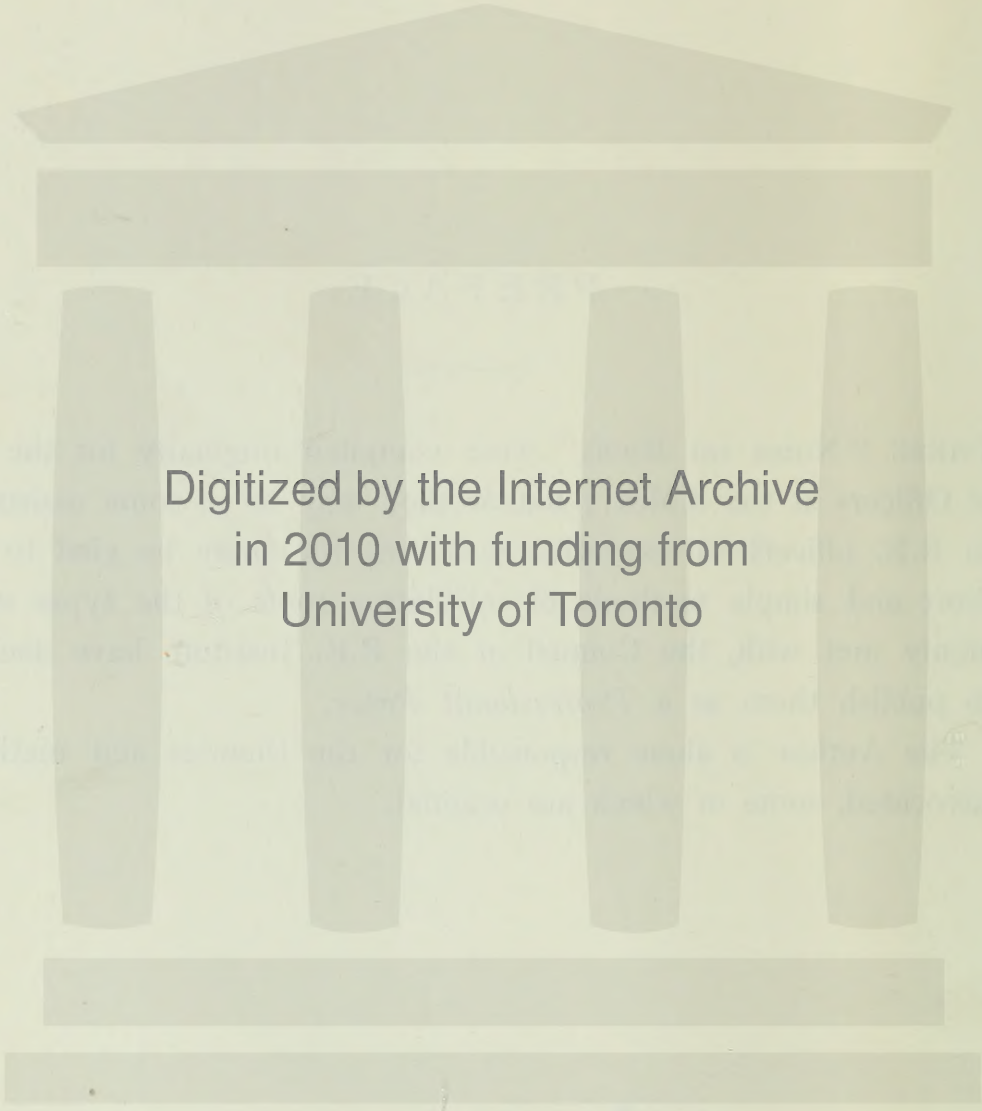
PREFACE.

THESE "Notes on Roofs" were compiled originally for the use of Officers at the S.M.E.; but as they may be of some assistance to R.E. officers in executive positions, who may be glad to use short and simple methods of calculating roofs of the types commonly met with, the Council of the R.E. Institute have decided to publish them as a *Professional Paper*.

The Author is alone responsible for the theories and methods advocated, some of which are original.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NOTES ON ROOFS.

I.—ROOF LOADS.

Introductory.

It is in the design of the roof more than in any other part of a building that a knowledge of Applied Mechanics can be usefully employed to restrict waste of material, whilst ensuring that a reasonable factor of safety is given.

In Military Buildings especially, much repetition of identical type occurs—for instance, in a row of barrack blocks one roof truss may be repeated perhaps a hundred times; time is in such cases well spent on calculation of the roof to ensure the maximum of economy compatible with safety.

In roof design the main basis of the calculation must be to a great extent empirical, and the result of calculation must necessarily be modified by practical requirements, so that great accuracy is out of place. It is the object of these notes to facilitate and simplify the calculation of roofs met with in ordinary daily practice.

Loads on Roofs.

The loads on roofs consist of :—

(*a*). *The permanent load* including the weight of the roof covering with battens or boarding, common rafters, purlins, and trusses; also the ceiling if there is one, and occasionally a lantern skylight, a train of punkahs, or other additional load which may be superimposed or suspended.

(*b*). *The occasional loads* which include wind pressure, workmen constructing or repairing the roof, and snow in certain latitudes; these it may be observed do not act concurrently; workmen will not be on a roof in a gale of wind, also except in very cold climates all the snow will be blown off a pitched roof before the full wind pressure can take effect.

Tables of Permanent Load.

The permanent load on a roof may be worked out by independent experiment or calculation, or may in most ordinary cases be taken from the tables following :—

TABLE I.

PERMANENT LOADS ON LIGHT TEMPORARY OR SEMI-PERMANENT ROOFS.

Nature of Roof Covering.	Usual Pitch, *i.e.*, Proportion of Rise to Span.	Slope of Roof.	Angle of Slope	Approximate Weight in lbs. per sq. ft. of Roof Surface	Approximate Weight in lbs per sq. ft. of Covered Area.
Corrugated iron, 20 or 22 S.W.G. ...	1/6	1/3	18° 25′	2	
Corrugated iron, 24 or 26 S.W.G. ...	1/6	1/3	18° 25′	1½	
Plain sheet iron, 22 or 24 S.W.G., Naini Tal or Le Mesurier system	1/4	1/2	26° 35′	2	
Eternit asbestos slates	1/4	1/2	26° 35′	2½	
Ruberoid, 3-ply	1/4	1/2	26° 35′	1	
Willesden paper or canvas	1/3	2/3	33° 42′	½–¾	
Asphalted roofing felt	1/3	2/3	33° 42′	½–¾	
Deal boarding, ¾″	—	—	—	2	
Deal boarding, 1″	—	—	—	2½	
Deal boarding, 1¼″...	—	—	—	3½	
Deal boarding, 1½″...	—	—	—	4	
Purlins (if any)	—	—	—	1	
Light timber trusses for spans up to 30′	—	—	—	—	2
Light steel trusses for spans 30′ to 60′...	—	—	—	—	3
Ceiling, ⅝″ matchboard	—	—	—	—	2

TABLE II.

PERMANENT LOADS ON ROOFS FOR PERMANENT BUILDINGS.

Nature of Roof Covering.	Usual Pitch, *i.e.*, Proportion of Rise to Span.	Slope of Roof.	Angle of Slope.	Approximate Weight in lbs. per sq. ft. of Roof Surface.	Approximate Weight in lbs per sq. ft. of Covered Area.
Best Welsh slates laid to 3″ lap ...	1/4	1/2	26° 35′	5	
Seconds do. do ...				6	
Thirds do. do. ...	2/7	4/7	29° 41′	7	
Best Westmoreland slates ...	1/3	2/3	33° 42′	9	
Seconds do.				11	
Thirds do.				14	
Plain tiles, 4″ gauge, 2¼″ lap ...	1/3	2/3	33° 42′	15	
	1/2	1/1	45° 0′		
	2/3	4/3	53° 0′		
Pantiles, Marseilles or Mangalore tiles	1/3	2/3	33° 42′	10	
	1/2	1/1	45° 0′		
Italian or single Allahabad tiles... ...	1/4	1/2	26° 35′	15	
Double Allahabad tiles	1/4	1/2	26° 35′	30	
Sarking felt or paper	—	—	—	½	
Battens for tiles or slates...	—	—	—	1	
Deal boarding, 1″	—	—	—	2½	
Common rafters	—	—	—	1	
Purlins	—	—	—	1	
Timber roof trusses for spans up to 30′...	—	—	—	—	3
Steel roof trusses for spans 30′ to 60′ ...	—	—	—	—	4
Ceiling of lath and plaster	—	—	—	—	10
Ceiling joists	—	—	—	—	2½

The snow load in this country does not amount to more than about 5 lbs. per square foot of area covered; but in Canada it may rise to as much as 30 lbs., and in the United States 15 lbs. is an average figure taken for roof calculations. In this country it is usually disregarded in calculations of pitched roofs on the supposition that it is not likely to act at the same time as the maximum wind pressure, or workmen's load. Snow.

In the calculations for flat roofs where snow may accumulate to some depth, and workmen might be employed to shovel the snow off the roofs, it will be desirable to make special allowance for snow in addition to the load for workmen.

The effect of the load due to workmen on a roof will vary with the nature of the roof covering and the pitch of the roof. On flat roofs the load may almost amount to as much as the load on a floor; on pitched roofs men move cautiously and distribute their weight by ladders or boards. Workmen.

A roof covering such as corrugated iron will serve to distribute the weight of the men over a large area, and the boarding usually laid under slates or tiles will answer the same purpose.

Taking these factors into consideration it appears that a suitable figure to allow for European workmen will be :—

15 lbs. per square foot of roof surface for light temporary or semi-permanent roofs.
20 lbs. per square foot of roof surface for roofs of asbestos slates.
25 lbs. " " " tiles or slates.

For bare-footed natives rather less will suffice.

The effect of wind pressure on roofs is somewhat indeterminate. The Board of Trade recommend that for bridges in exposed situations a maximum horizontal pressure of 56 lbs. per square foot of surface should be provided for. Wind pressure close to the ground is never so great owing to the retarding influence of the earth surface, and in no case will a roof be so exposed as a high bridge. A general rule in use for enclosure walls and chimneys is to take horizontal wind pressure in lbs. at any height from the ground $= 18 + \frac{\text{height in feet}}{4}$. Wind Pressure.

Another simpler rule is to take horizontal wind pressure = 1 lb. for each foot of height subject to a maximum of 50 lbs.

The London County Council have recently adopted a rule which allows 28 lbs. vertical load per square foot of sloping roof surface for all roofs of over 20° inclination.

The majority of military buildings are of no great height above the ground; but they are in some cases built on very exposed sites: the amount of wind pressure to be taken must therefore be a matter of judgment.

It is suggested that in ordinary cases it will be sufficient to take a

horizontal wind pressure of 40 lbs. which will give a component normal to the roof :—

For roofs sloping	1/3 = 18° 25′	of	17	lbs.
„ „	1/2 = 26° 35′	„	24	„
„ „	4/7 = 29° 41′	„	26	„
„ „	2/3 = 33° 42′	„	29	„
„ „	1/1 = 45° 0′	„	36	„
„ „	3/2 = 53° 0′	„	38	„

(For other assumptions as to wind pressure in proportion).

This load is taken as a dead load acting normal to the roof slope and may be regarded as acting on either side of the roof and resisted by parallel inclined reactions at both walls, except in the case of steel roofs where owing to the expansion arrangements the free end may only offer a vertical reaction.

General Summary.

The foregoing remarks may now be summarized. It will be seen that however carefully the exact weight of the roof covering and timbering may be taken out, the main factors in the calculation of roofs are the more or less arbitrarily assumed figures for load of workmen and wind pressure.

These loads act most abruptly on the battens or boarding immediately supporting the roof covering, and are transmitted more gradually in turn to the common rafters, purlins, and trusses. It will therefore be theoretically sound as well as practically convenient to work the stress in all members making up the roof with one figure for total roof load instead of following the usual academical process of taking the supposed superimposed load and the weight of roof covering and then working downwards through battens, rafters, and purlins to trusses, adding on the weight of each in turn.

Rules for Loads.

The following simple rules for roof loads will cover the more ordinary cases, and their application in the examples will show the method to adopt when for any reason other assumptions as to loading appear desirable :—

TABLE III.

Pitched Roofs.

WORKING LOADS FOR COMMON CASES OF SMALL SPAN.

	Weight of Roof Covering with Battens and Boarding if Required	Rafters, Purlins, and Trusses.	Add for Occasional Loads, *i.e.*, Workmen or Wind.	Total Working Load per sq. ft. of Roof Surface.
A.—Temporary roofs :— Such as corrugated iron, Willesden canvas or felt	2	3	15	**20**
B.—Light roofs :— Such as asbestos slates on 1″ boarding ...	6	4	20	**30**
C.—Ordinary roofs :— Such as Welsh slates on 1″ boarding ...	10	5	25	**40**
D.—Heavy roofs :— Such as thick Westmoreland slates or plain tiles on battens over felt and 1″ boarding	19	6	25	**50**
E.—Double roofs :— Such as double Allahabad tiling on battens	33	7	20	**60**

Add for lath and plaster ceilings if any, and for floor loads of attics or lofts if borne by roof. Ceilings.

Disregard snow unless in exceptional cases. Snow.

In exposed situations for all cases, and generally when the span exceeds 30′ or the slope of the roof is greater than 45°, work out the stresses due to wind pressure and add to those due to permanent load instead of the equivalent dead weight allowance for occasional load if they come out greater. Wind.

For flat roofs such as asphalt, or vulcanite, which are usually laid at a slope of 1 in 40, on concrete with steel joists or reinforcement; or lead, zinc, or copper, usually laid at a slope of 1 in 80, on boarding and timber; take out in detail the dead load of the roof, add for accumulated snow, and such load for work people and others as may appear necessary. Flat Roofs.

The London County Council rule is to add 56 lbs. vertical load per f.s. for all flat roofs over and above the actual dead weight of material.

In such roofs the possible use of the roof for temporarily stacking materials for the repair of adjacent pitched roofs, or even as a stand for spectators must be borne in mind so that any general rule as to safe load needs revision according to the circumstances of each case.

Wind pressure on flat roofs may be disregarded.

II.—DATA FOR ROOF CALCULATIONS.

Safe Stresses in Timber.

For timber a factor of safety of 5 is usually given and the following safe working stresses may be taken :—

TABLE IV.

WORKING STRESSES ON TIMBER.

Name of Timber.	Transverse stress r	Tension r_t	Compressn. r_c	Bearing r_b	Shearing across grain r_s	Shearing along grain r_l	Modulus of Elasticity E
	Safe stresses in lbs. per square inch.						
Baltic Fir	1200	1400	1000	1340	360	140	1,440,000
Pitch Pine	1400	1120	1120	1400	—	—	1,500,000
British Oak	1600	1750	1450	2800	560	224	1,200,000
Deodar (India)	1008	700	700	700	—	—	1,500,000
Teak (Burmah)	1530	1210	1340	1340	—	—	1,600,000
Sal (India)	1640	1210	1210	1210	—	—	1,800,000

For other timbers and for the figures on which these average working stresses are based see

Scott-Moncrieff's *Structural Design*, Part I., Chapter III.
Rivington's *Building Construction*, Part IV., Tables I. and IA.
Seddon's *Builder's Work*, Appendix VI.
Molesworth's Pocket Book.
Military Works Handbook, India.
etc. *etc.*

It must be noted that confusing and contradictory figures are given even side by side in the same books, and that much reliance is not to be placed on results obtained by breaking small selected test pieces.

In case of doubtful or unknown timber it may be best to deduce the value of r for transverse stress from experiment by breaking a few average scantlings, and the value of E from actual deflection tests which may be made without any elaborate apparatus ; reasonable working stresses may then be assumed for all other conditions.

Formulæ for calculation of battens, purlins, etc., under transverse load : it is usual to take such timbers as supported only at the points on which they rest, as a matter of fact they are often continuous over three or more supports and an extra factor of safety is thus given ; but in cutting and replacing defective timber short lengths may be used and it is therefore safer to assume support only at each bearing. Transverse Stress.

For a beam supported at each end and uniformly loaded the maximum moment of flexure $M_f = \frac{Wl}{8}$ and the moment of resistance for a rectangular beam $M_r = \frac{rbd^2}{6}$.

Equating $M_f = M_r$

$$\frac{Wl}{8} = \frac{rbd^2}{6}.$$

W=total dead distributed load on the beam in lbs.
r=safe transverse stress in lbs. per square inch.
bd and l=breadth, depth and length all in inches.

For Baltic fir r may be taken=1,200 lbs. per square inch, *i.e.*, allowing for a breaking stress of 6,000 lbs. with factor of safety of 5 ; and as it is usually more convenient to measure the length of bearing in feet we may substitute L=length in feet $=\frac{l}{12}$.

The formula for fir beams supported and uniformly loaded may then be conveniently written

$$bd^2 = \frac{Wl \times 6}{8 \times r} = \frac{WL \times 12 \times 6}{8 \times 1200} = \frac{WL}{133}.$$

Where b and d=breadth and depth in inches
L=clear span in FEET
W=total dead distributed load in lbs.

That is if the bd^2 of the beam is $> \frac{WL}{133}$ it is strong enough.

The general formula for deflection is Deflection.

$$\Delta = \frac{nWl^3}{EI} \text{ or } = \frac{n^1 r l^2}{Ey}.$$

Where Δ=deflection in inches.
n=a co-efficient varying with the loading and method of support (see table of n and n^1 below).
W=total load on the beam.
l=length of beam in inches.
E=modulus of elasticity in the same unit as W.
I=moment of inertia of the section.
r=stress in lbs. per square inch at the extreme fibre.
y=distance in inches of extreme fibre from neutral axis.

Arrangement of Beam and Load.	Beam of uniform Cross Section throughout its length.	
	Value of n	Value of n^1
Supported at both ends and loaded uniformly	$\frac{5}{384}$	$\frac{5}{48}$
Supported at both ends and loaded at centre	$\frac{1}{48}$	$\frac{1}{12}$
Fixed at both ends and loaded uniformly	$\frac{1}{384}$	$\frac{1}{16}$
Fixed at both ends and loaded at centre	$\frac{1}{192}$	$\frac{1}{24}$

Then for a beam of Baltic fir uniformly loaded and supported at each end

$$n=\frac{5}{384}.$$

$$\mathrm{E}=1{,}440{,}000.$$

$$\mathrm{I}=\frac{bd^3}{12}.$$

As before we may substitute L for $\frac{l}{12}$ and the allowable deflection in roof timbers where deflection need be considered, *i.e.*, in the purlins or boarding or battens carrying a thin slate roof, or in the ceiling rafters carrying a plaster ceiling is $\frac{1}{40}''$ per foot of span.

$\therefore$ Deflection in inches $=\frac{\mathrm{L}}{40}$.

The formula may thus be simplified :—

$$\Delta=\frac{n\mathrm{W}l^3}{\mathrm{EI}}.$$

$$\frac{\mathrm{L}}{40}=\frac{5\times\mathrm{W}\times12\times12\times12\times\mathrm{L}^3\times12}{384\times1{,}440{,}000\times bd^3}$$

or $$bd^3=\frac{\mathrm{WL}^2}{133}.$$

Where b and d = breadth and depth in inches.
L = clear span in FEET.
W = total dead distributed load in lbs.

That is if the bd^3 of the beam is $>\frac{\mathrm{WL}^2}{133}$ it is stiff enough.

Similar simple formulæ for practical use may be deduced for the other timbers commonly used for roof work and are tabulated below with those for transverse stress.

TABLE V.

TABLE OF FORMULÆ FOR TIMBER BEAMS UNIFORMLY LOADED AND SUPPORTED AT EACH END.

Nature of Timber.	TRANSVERSE STRESS. Factor of Safety = 5.	DEFLECTION. $\frac{1}{360}$" per foot run.
Baltic Fir	$bd^2 = \frac{WL}{133}$	$bd^3 = \frac{WL^2}{133}$
Pitch Pine	$bd^2 = \frac{WL}{156}$	$bd^3 = \frac{WL^2}{139}$
British Oak	$bd^2 = \frac{WL}{178}$	$bd^3 = \frac{WL^2}{111}$
Deodar (India)	$bd^2 = \frac{WL}{112}$	$bd^3 = \frac{WL^2}{139}$
Teak (Burmah)	$bd^2 = \frac{WL}{170}$	$bd^3 = \frac{WL^2}{148}$
Sal (India)	$bd^2 = \frac{WL}{182}$	$bd^3 = \frac{WL^2}{167}$

Where b and d = breadth and depth in inches.

L = clear span in FEET.
W = total dead distributed load in lbs.

The values of bd^2 or bd^3 being found for any particular case suitable size of timber can be readily selected from the tables of bd^2 and bd^3 which follow, giving suitable proportions to b and d.

For example, purlins on a roof sloping 1 in 1 = 45° will usually be made square, whilst on a roof at slope 1 in 2 = 26° 33′ a proportion of $b:d::2:3$ would be better.

For beams with one side vertical or nearly so a proportion $b:d::1:2$ will be suitable, or even deeper timber may be in some cases used when the beams are well fixed to prevent lateral motion as in the case of rafters with boarding nailed across them, or the parallel case of floor joists suitably strutted.

TABLE VI.—VALUES OF bd^2, b VARYING FROM 1 TO 6, d VARYING FROM 1 TO 12.

DEPTH.	BREADTH.																			
	1	1¼	1½	1¾	2	2¼	2½	2¾	3	3¼	3½	3¾	4	4¼	4½	4¾	5	5¼	5½	5¾	6
1	1·0	1·2	1·5	1·7	2·0	2·2	2·5	2·7	3·0	3·2	3·5	3·7	4·0	4·2	4·5	4·7	5·0	5·2	5·5	5·7	6·0
1¼	1·6	2·0	2·3	2·7	3·1	3·5	3·9	4·3	4·7	5·1	5·5	5·9	6·2	6·6	7·0	7·4	7·8	8·2	8·6	9·0	9·4
1½	2·2	2·8	3·4	3·9	4·5	5·1	5·6	6·2	6·7	7·3	7·9	8·4	9·0	9·6	10·1	10·7	11·2	11·8	12·4	12·9	13·5
1¾	3·1	3·8	4·6	5·4	6·1	6·9	7·7	8·4	9·2	10·0	10·7	11·5	12·3	13·0	13·8	14·6	15·3	16·1	16·8	17·6	18·4
2	4·0	5·0	6·0	7·0	8·0	9·0	10·0	11·0	12·0	13·0	14·0	15·0	16·0	17·0	18·0	19·0	20·0	21·0	22·0	23·0	24·0
2¼	5·1	6·3	7·6	8·9	10·1	11·4	12·7	13·9	15·2	16·2	17·7	19·0	20·2	21·5	22·8	24·0	25·3	26·6	27·8	29·1	30·4
2½	6·2	7·8	9·4	10·9	12·5	14·1	15·6	17·2	18·7	20·3	21·9	23·4	25·0	26·6	28·1	29·7	31·2	32·8	34·4	35·9	37·5
2¾	7·6	9·5	11·3	13·2	15·1	17·0	18·9	20·8	22·7	24·6	26·5	28·4	30·2	32·1	34·0	35·9	37·8	39·7	41·6	43·5	45·4
3	9·0	11·2	13·5	15·7	18·0	20·2	22·5	24·7	27·0	29·2	31·5	33·7	36·0	38·2	40·5	42·7	45·0	47·2	49·5	51·7	54·0
3¼	10·6	13·2	15·8	18·5	21·1	23·8	26·4	29·0	31·7	34·3	37·0	39·6	42·2	44·9	47·5	50·2	52·8	55·5	58·1	60·7	63·4
3½	12·2	15·3	18·4	21·4	24·5	27·6	30·6	33·7	36·7	39·8	42·9	45·9	49·0	52·1	55·1	58·2	61·2	64·3	67·4	70·4	73·5
3¾	14·1	17·6	21·1	24·6	28·1	31·6	35·2	38·0	42·2	45·7	49·2	52·7	56·2	59·8	63·3	66·8	70·2	73·8	77·3	80·9	84·4
4	16·0	20·0	24·0	28·0	32·0	36·0	40·0	44·0	48·0	52·0	56·0	60·0	64·0	68·0	72·0	76·0	80·0	84·0	88·0	92·0	96·0
4¼	18·1	22·6	27·1	31·6	36·1	40·6	45·2	49·7	54·2	58·7	63·2	67·0	72·2	76·8	81·3	85·8	90·3	94·8	99·3	104	108
4½	20·2	25·3	30·4	35·4	40·5	45·6	50·6	55·7	60·7	65·8	70·9	75·9	81·0	86·1	91·1	96·2	101	106	111	116	121
4¾	22·6	28·2	33·8	39·5	45·1	50·8	56·4	62·0	67·7	73·3	79·0	84·6	90·2	95·9	102	107	113	118	124	130	135
5	25·0	31·2	37·5	43·7	50·0	56·2	62·5	68·7	75·0	81·2	87·5	93·7	100	106	112	119	125	131	137	144	150
5¼	27·6	34·5	41·3	48·2	55·1	62·0	68·9	75·8	82·7	89·6	96·5	103	110	117	124	131	138	145	152	158	165
5½	30·2	37·8	45·4	52·9	60·5	68·1	75·6	83·2	90·7	98·3	106	113	121	129	136	144	151	159	166	174	181
5¾	33·1	41·3	49·6	57·9	66·1	74·4	82·7	90·9	99·2	107	116	124	132	141	149	157	165	174	182	190	198
6	36·0	45·0	54·0	63·0	72·0	81·0	90·0	99·0	108	117	126	135	144	153	162	171	180	189	198	207	216
6¼	39·1	48·8	58·6	68·4	78·1	87·9	97·0	107	117	127	137	146	156	166	176	186	195	205	215	225	234
6½	42·2	52·8	63·4	73·9	84·5	95·1	106	116	127	137	148	158	169	180	190	201	211	222	232	243	253
6¾	45·6	57·0	68·3	79·7	91·1	103	114	125	137	148	159	171	182	194	205	216	228	239	251	262	273
7	49·0	61·2	73·5	85·7	98·0	110	122	135	147	159	171	184	196	208	220	233	245	257	269	282	294
7¼	52·6	65·7	78·9	92·0	105	118	131	145	158	171	184	197	210	223	237	250	263	276	289	302	315
7½	56·2	70·3	84·4	98·4	112	127	141	155	169	183	197	211	225	239	253	267	281	295	309	323	337
7¾	60·1	75·1	90·1	105	120	135	150	165	180	195	210	225	240	255	270	285	300	315	330	345	360
8	64·0	80·0	96·0	112	128	144	160	176	192	208	224	240	256	272	288	304	320	336	352	368	384
8¼	68·1	85·1	102	119	136	153	170	187	204	221	238	255	272	289	306	323	340	357	374	391	408
8½	72·2	90·3	108	126	144	163	181	199	217	235	253	271	289	307	325	343	361	379	397	415	433
8¾	76·6	95·7	115	134	153	172	191	211	230	249	268	287	306	325	345	364	383	402	421	440	459
9	81·0	101	121	142	162	182	202	223	243	263	283	304	324	344	364	385	405	425	445	466	486
9¼	85·6	107	128	150	171	193	214	235	257	278	299	321	342	364	385	406	428	449	470	492	513
9½	90·2	113	135	158	180	203	226	248	271	293	316	338	361	384	406	429	451	474	496	519	541
9¾	95·1	119	142	166	190	214	238	261	285	309	333	356	380	404	428	452	475	499	523	547	570
10	100	125	150	175	200	225	250	275	300	325	350	375	400	425	450	475	500	525	550	575	600
11	121	151	181	212	242	272	302	333	363	393	423	454	484	514	544	575	605	635	665	696	726
12	144	180	216	252	288	324	360	396	432	468	504	540	576	612	648	684	720	756	792	828	864

TABLE VII. — Values of bd^3, b varying from 1 to 6, d varying from 1 to 12.

Depth.	Breadth.																		
	1	1¼	1½	1¾	2	2¼	2½	2¾	3	3¼	3½	3¾	4	4¼	4½	4¾	5	5¼	5½	5¾	6
1	1·0	1·2	1·5	1·7	2·0	2·2	2·5	2·7	3·0	3·2	3·5	3·7	4·0	4·2	4·5	4·7	5·0	5·2	5·5	5·7	6·0
1¼	2·0	2·4	2·9	3·4	3·9	4·4	4·9	5·4	5·9	6·3	6·8	7·3	7·8	8·3	8·8	9·3	9·8	10·3	10·7	11·2	11·7
1½	3·4	4·2	5·1	1·9	6·7	7·6	8·4	9·3	10·1	11·0	11·8	12·7	13·5	14·3	15·2	16·0	16·9	17·7	18·6	19·4	20·2
1¾	5·4	6·7	8·0	9·4	10·7	12·1	13·4	14·7	16·1	17·4	18·8	20·1	21·4	22·8	24·1	25·5	26·8	28·1	29·5	30·8	32·2
2	8·0	10·0	12·0	14·0	16·0	18·0	20·0	22·0	24·0	26·0	28·0	30·0	32·0	34·0	36·0	38·0	40·0	42·0	44·0	46·0	48·0
2¼	11·4	14·2	17·1	19·9	22·8	25·6	28·5	31·3	34·2	37·0	39·9	42·7	45·6	49·4	51·3	54·1	57·0	59·8	62·6	65·5	68·3
2½	15·6	19·5	23·4	27·3	31·2	35·2	39·1	43·0	46·9	50·8	54·7	58·6	62·5	66·4	70·3	74·2	78·1	82·0	85·9	89·8	93·7
2¾	20·8	26·0	31·2	36·4	41·6	46·8	52·0	57·2	62·4	67·6	72·8	78·0	83·2	88·4	93·6	98·8	104	109	114	120	125
3	27·0	33·7	40·5	47·2	54·0	60·7	67·5	74·2	81·0	87·7	94·5	101	108	115	121	128	135	142	148	155	162
3¼	34·3	42·9	51·5	60·1	68·7	77·2	85·8	94·4	103	112	120	129	137	146	154	163	172	180	189	197	206
3½	42·9	53·6	64·3	75·0	85·8	96·5	107	118	129	139	150	161	171	182	193	204	214	225	236	247	257
3¾	52·7	65·9	79·1	92·3	105	119	132	145	158	171	185	198	211	224	237	250	264	277	290	303	316
4	64·0	80·0	96·0	112	128	144	160	176	192	208	224	240	256	272	288	304	320	336	352	368	384
4¼	76·8	96·0	115	134	154	173	192	211	230	249	269	288	307	326	345	365	384	403	422	441	461
4½	91·1	114	137	159	182	205	228	251	273	296	319	342	364	387	410	433	456	478	501	524	547
4¾	107	134	161	188	214	241	268	295	322	348	375	402	429	455	482	509	536	563	589	616	643
5	125	156	187	219	250	281	312	344	375	406	437	469	500	531	562	594	625	656	687	719	750
5¼	145	181	217	253	289	326	362	398	434	470	506	543	579	615	651	687	724	760	796	832	868
5½	166	208	250	291	333	374	416	458	499	541	582	624	665	707	749	790	832	873	915	957	998
5¾	190	238	285	333	380	428	475	523	570	618	665	713	760	808	855	903	951	998	1046	1093	1141
6	216	270	324	378	432	486	540	594	648	702	756	810	864	918	972	1026	1080	1134	1188	1242	1296
6¼	244	305	366	427	488	549	610	671	732	793	855	916	977	1038	1099	1160	1221	1282	1343	1404	1465
6½	275	343	412	481	549	618	687	755	824	893	961	1030	1098	1167	1236	1304	1373	1442	1510	1579	1648
6¾	308	384	461	538	615	692	769	846	923	1000	1076	1153	1230	1307	1384	1461	1538	1615	1692	1768	1845
7	343	429	514	600	686	772	857	943	1029	1115	1200	1286	1372	1458	1543	1629	1715	1801	1886	1972	2058
7¼	381	476	572	667	762	857	953	1048	1143	1238	1334	1429	1524	1620	1715	1810	1905	2001	2096	2191	2286
7½	422	527	633	738	844	949	1055	1160	1266	1371	1477	1582	1687	1793	1898	2004	2109	2215	2320	2426	2531
7¾	465	582	698	815	931	1047	1164	1280	1396	1513	1629	1746	1862	1978	2095	2211	2327	2444	2560	2677	2793
8	512	640	768	896	1024	1152	1280	1408	1536	1664	1792	1920	2048	2176	2304	2432	2560	2688	2816	2944	3072
8¼	562	702	843	983	1123	1263	1404	1544	1685	1825	1965	2106	2246	2386	2527	2667	2808	2948	3088	3229	3369
8½	614	768	921	1075	1228	1382	1535	1689	1842	1996	2149	2303	2456	2610	2763	2917	3071	3224	3378	3531	3685
8¾	670	837	1005	1172	1340	1507	1675	1842	2010	2177	2345	2512	2680	2847	3015	3182	3350	3517	3685	3852	4020
9	729	911	1093	1276	1458	1640	1822	2005	2187	2369	2551	2734	2916	3098	3280	3463	3645	3827	4009	4192	4374
9¼	791	989	1187	1385	1583	1781	1979	2176	2374	2572	2770	2968	3166	3364	3562	3759	3957	4155	4353	4551	4749
9½	857	1072	1286	1500	1715	1929	2143	2358	2572	2786	3001	3215	3429	3644	3858	4073	4289	4501	4716	4930	5144
9¾	927	1159	1390	1622	1854	2085	2317	2549	2781	3012	3244	3476	3707	3939	4171	4403	4634	4866	5098	5329	5561
10	1000	1250	1500	1750	2000	2250	2500	2750	3000	3250	3500	3750	4000	4250	4500	4750	5000	5250	5500	5750	6000
11	1331	1664	1997	2329	2662	2995	3328	3660	3993	4326	4659	4991	5324	5657	5990	6322	6655	6988	7321	7653	7986
12	1728	2160	2592	3024	3456	3888	4320	4752	5184	5616	6048	6480	6912	7344	7776	8208	8640	9072	9504	9936	10368

Timber Struts.

In applying the working stress in compression (Table IV.) to timber struts, whether in direct compression only as in the case of the struts in a king post truss, or in combined stress consisting partly of direct compression stress and partly of transverse stress as in case of a roof rafter, it is necessary to take into account the proportion of the length of a strut to its least dimension and to reduce the working stress applied to the material as may be necessary to avoid the risk of the member buckling.

For timber compression members in roofs fixed in the usual way the following empirical method of reduction of stress is a simple one for practical work.

TABLE VIII.

SAFE STRESS IN STRUTS OF BALTIC FIR.

Proportion of Length to Least Dimension.	Safe Stress in lbs. per square inch.
$\frac{l}{h} = 8$	1000
$\frac{l}{h} = 10$	800
$\frac{l}{h} = 16$	600
$\frac{l}{h} = 24$	400
$\frac{l}{h} = 36$	200
$\frac{l}{h} = 60$	100

Intermediate values by interpolation.

Other timbers in proportion of value r_c: 1,000 lbs. for fir (Table IV.).

Timber Ties.

In timber ties the working stress in tension (Table IV.) gives the size necessary; but bolt holes and notches at joints must be deducted in taking the section available.

For steel and wrought iron a factor of safety of 4 is usually given and the following safe working stresses may be taken :— Safe Stresses in Steel and Iron.

TABLE IX.

WORKING STRESSES ON IRON AND STEEL.

Material.	Transverse Stress r	Tension r_t	Compression r_c	Bearing r_b	Shearing r_s	Modulus of Elasticity.
	Safe Stress in Tons per square inch.					Lbs. per sq. inch.
Steel	$7\frac{1}{2}$	8	7	8	$5\frac{1}{2}$	30,000,000
Wrought Iron	5	5	5	6	4	29,000,000
Cast Iron	$3\frac{1}{2}$	$1\frac{1}{2}$	8	10	$2\frac{1}{2}$	17,000,000

Formulæ for the calculation of purlins or other beams under transverse load: for the common case considered as a beam supported at each end and loaded uniformly :— Transverse Stress.

$$M_{ff}=\frac{Wl}{8}=M_r=\frac{rI}{y}.$$

And taking span for convenience in feet $L=\frac{l}{12}$,

and r for steel $=7\frac{1}{2}$ tons per square inch.

$$\therefore \quad \frac{I}{y}=\frac{Wl}{8r}=\frac{WL\times 12\times 2}{8\times 15}=\frac{WL}{5}.$$

For all purlins of steel whether of angles, channels or rolled steel beams we thus have a simple formula :—

$$\frac{I}{y}=\frac{WL}{5}.$$

Where $\frac{I}{y}$ = section modulus (or moment of resistance at 1 ton per square inch), taken from tables of British Standard Sections.

W = total distributed load in TONS.

L = clear span in FEET.

In cases where the beam is continuous over several supports it is safe to take a lower section modulus in proportion as the bending moment is reduced, viz., $::\frac{Wl}{12}:\frac{Wl}{8}$.

As in the case of timber beams the general formula for deflection may be simplified for the case of a steel beam of any section supported at each end and loaded uniformly. Deflection.

The formula—

$$\Delta = \frac{nWl^3}{EI}$$

becomes $$\frac{L}{40} = \frac{5 \times W \times L^3 \times 12 \times 12 \times 12 \times 2240}{384 \times 30{,}000{,}000 \times I}$$

or $$I = \frac{5 \times W \times L^3 \times 12 \times 12 \times 12 \times 40 \times 2240}{384 \times 30{,}000{,}000 \times L}$$

or $$I = \frac{WL^2}{15} \text{ nearly.}$$

So for deflection we thus have a simple formula :—

$$I = \frac{WL^2}{15}.$$

Where I = moment of inertia (from Tables of British Standard Sections).

W = total distributed load in TONS.

L = clear span in FEET.

In cases where the beam is continuous over several supports it is safe to take a lower moment of inertia in proportion as the deflection is reduced, viz., ::1:5 (*vide* table of value of n in general deflection formula).

Steel or Iron Struts.

In the compression bars of steel or iron roofs it is necessary as in the case of timber struts to allow for a reduction of stress to avoid the risk of buckling.

This reduction is made empirically by taking into account the proportion of the unsupported length of the bar to the least radius of gyration of the section.

The radius of gyration $= \sqrt{\frac{I}{A}}$.

Where I is the least moment of inertia

A = the area of the section.

It is somewhat laborious to work out the value of the radius of gyration; but it may in most cases be taken from the Tables of British Standard Sections, or obtained by the use of the factor n shown below Table X., which gives the approximate value directly from the dimensions of the section.

For practical purposes the following tables which shows the safe stress which may be applied to compression bars of iron or steel for various proportions of $l:k$ may be used :—

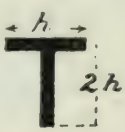
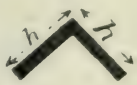
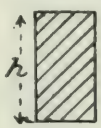
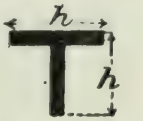
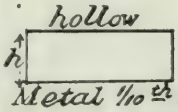
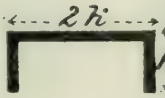
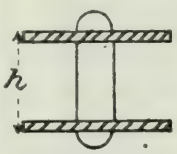
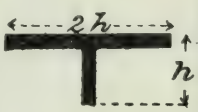
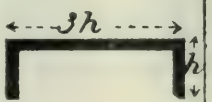
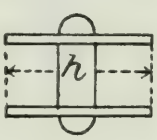
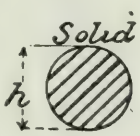
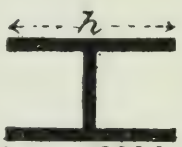
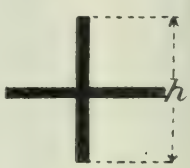
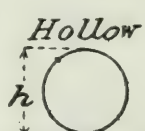
TABLE X.

SAFE STRESS IN COLUMNS OF CAST IRON, WROUGHT IRON, AND STEEL.

l = length of strut between ends in inches.
k = least radius of gyration, *i.e.*, the square root of Moment of Inertia ÷ area of cross section.
r_c = safe stress in tons per square inch.

$\frac{l}{k}$	r_c						$\frac{l}{k}$
	Cast Iron.		Wrought Iron.		Mild Steel.		
	Ends Rounded.	Ends Fixed.	Ends Rounded.	Ends Fixed.	Ends Rounded.	Ends Fixed.	
10	8·68	8·85	4·00	4·00	5·33	5·34	10
15	8·41	8·76	3·98	4·00	5·26	5·31	15
20	8·07	8·65	3·92	3·99	5·20	5·29	20
25	7·58	8·46	3·88	3·98	5·13	5·24	25
30	6·98	8·21	3·80	3·95	5·02	5·20	30
35	6·32	7·91	3·72	3·92	4·90	5·15	35
40	5·68	7·56	3·64	3·89	4·76	5·09	40
45	5·02	7·19	3·54	3·86	4·58	5·03	45
50	4·43	6·82	3·44	3·82	4·40	4·98	50
55	3·84	6·46	3·31	3·78	4·22	4·92	55
60	3·35	6·10	3·17	3·73	4·02	4·83	60
65	2·92	5·75	3·04	3·68	3·80	4·75	65
70	2·57	5·39	2·90	3·63	3·59	4·67	70
75	2·23	5·02	2·76	3·55	3·37	4·56	75
80	1·96	4·68	2·60	3·48	3·15	4·45	80
85	1·74	4·33	2·46	3·40	2·96	4·35	85
90	1·56	4·00	2·33	3·32	2·77	4·25	90
95	1·42	3·66	2·18	3·22	2·56	4·13	95
100	1·29	3·35	2·03	3·17	2·40	4·00	100
105	1·17	3·07	1·92	3·08	2·24	3·88	105
110	1·07	2·80	1·79	3·00	2·08	3·74	110
115	0·99	2·57	1·67	2·91	1·95	3·61	115
120	0·93	2·37	1·57	2·82	1·83	3·46	120
125	0·86	2·19	1·47	2·74	1·71	3·32	125
130	0·80	2·03	1·39	2·66	1·61	3·21	130
135	0·75	1·90	1·32	2·58	1·50	3·09	135
140	0·70	1·78	1·24	2·48	1·42	2·96	140
145	0·66	1·66	1·17	2·40	1·36	2·85	145
150	0·61	1·56	1·10	2·32	1·28	2·72	150
160	0·56	1·40	0·98	2·14	1·13	2·51	160
170	0·49	1·25	0·88	2·00	1·01	2·32	170
180	0·43	1·14	0·80	1·84	0·91	2·13	180
190	0·39	1·03	0·72	1·70	0·83	1·97	190
200	0·36	0·93	0·66	1·57	0·75	1·83	200
210	0·32	0·84	0·58	1·46	0·68	1·68	210
220	0·30	0·77	0·55	1·35	0·62	1·55	220
230	0·28	0·70	0·50	1·26	0·58	1·44	230
240	0·25	0·64	0·46	1·18	0·53	1·34	240
250	0·23	0·59	0·42	1·11	0·49	1·25	250
260	0·22	0·56	0·40	1·04	0·46	1·16	260
270	0·20	0·52	0·37	0·97	0·42	1·08	270
280	0·19	0·49	0·35	0·91	0·39	1·01	280
290	0·18	0·46	0·32	0·86	0·37	0·96	290
300	0·17	0·43	0·30	0·80	0·35	0·92	300

For practical purposes the value of $\frac{l}{k}$ can be found with sufficient accuracy from $\frac{l}{k} = n\frac{l}{h}$, where n and h are taken as below :—

Section.	n.	Section.	n.	Section.	n.
	3·5		6·0		4·9
	2·0		4·9		3·9
	2·5				3·8
	3·0		4·2		4·3
	2·0		3·9		
	4·0		4·2		4·9
	3·1				

Joints.

In practice more roofs fail at the joints than elsewhere, it is therefore necessary to see that the joints are capable of taking the full amount of stress which the members can bear.

Bolts and Straps.

A table giving the shearing and bearing values of bolts or nails and spikes in timber for the joints of timber roofs has been compiled below to save labour in calculation ; the figures in this case being given in lbs., and the stresses taken a little below the usual working limits to give extra safety in the joints.

TABLE XII.—Shearing and Bearing Value of Nails, Bolts, and Spikes in Lbs.

Size of Nail, Bolt, or Spike.		Area in sq. in.	Shearing Value at 8,000 lbs. = 3·57 tons per sq. in.		Bearing Value in Baltic Fir at 1,340 lbs. per sq. in. Thickness of Timber in inches.									
			Single	D'ble.	1½	2	2½	3	3½	4	4½	5	5½	6
3″ Wire Nails, 8 S.W.G. = 0·160		0·020	161	281	322	429	536	643	750	857	965	1072	1179	1286
4″ Wire Nails, 6 S.W.G. = 0·192		0·029	232	405	386	515	643	772	900	1029	1158	1286	1415	1544
5″ Wire Nails, 5 S.W.G. = 0·212		0·035	282	494	426	568	715	852	994	1136	1278	1420	1562	1704
6″ Wire Nails, 4 S.W.G. = 0·232		0·042	338	592	466	622	777	933	1088	1243	1399	1554	1710	1865
Spikes or Bolts.	¼″	0·049	393	687	502	670	837	1005	1172	1340	1507	1675	1842	2010
	⅜″	0·110	883	1546	754	1005	1256	1507	1759	2010	2261	2512	2764	3015
	½″	0·196	1570	2748	1005	1340	1675	2010	2345	2680	3015	3350	3685	4020
	⅝″	0·307	2454	4295	1256	1675	2093	2512	2941	3350	3769	4187	4606	5025
	¾″	0·442	3536	6184	1507	2010	2512	3015	3517	4020	4522	5025	5527	6030
	1″	0·785	6283	10996	2010	2680	3350	4020	4690	5360	6030	6700	7370	8040
	1¼″	1·227	9816	17178	2512	3350	4187	5025	5 862	6700	7537	8375	9212	10050
	1½″	1·767	14136	24738	3015	4020	5025	6030	7035	8040	9045	10050	11055	12060
	1¾″	2·405	19240	33670	3517	4690	5862	7035	8207	9380	10552	11725	12897	14070
	2″	3·142	25136	43988	4020	5360	6700	8040	9380	10720	12060	13400	14740	16080

Bearing Equivalents on Wrought Iron Straps at 13,000 lbs. = 5·81 tons per sq. in.

Note. The strap must be wide enough to take the required tension at 11,400 lbs. per sq. in. after deducting the bolt-hole, and in no case of less width than 3 × diam. of bolt, unless a forged eye is used. The centre of bolt hole must be not less than 2 × diam. from end of strap.

Size of Nail, Bolt, or Spike.		Single Straps.					Double Straps.				
		¼″	⅜″	½″	⅝″	¾″	¼″	⅜″	½″	⅝″	¾″
Spikes or Bolts.	¼″										
	⅜″	1219	1828	2437	3046	3656	2132	3199	4265	5332	6398
	½″	1625	2437	3250	4062	4875	2843	4265	5687	7109	8531
	⅝″	2031	3047	4062	5078	6093	3554	5332	7109	8886	10664
	¾″	2437	3656	4875	6093	7312	4265	6398	8531	10664	12797
	1″	3250	4875	6500	8125	9750	5687	8531	11375	14219	17062
	1¼″	4062	6093	8125	10156	12187	7109	10664	14218	17773	21327
	1½″	4875	7312	9750	12187	14625	8531	12797	17062	21327	25593
	1¾″	5687	8531	11375	14218	17062	9953	14930	18906	24882	29858
	2″	6500	9750	13000	16250	19500	11375	17062	22750	28437	34123

In designing joints and using this table the following points should be taken into consideration :—

With bolts or spikes in timber the chief weakness is the shearing of the wood along the grain. To guard against this the nails, bolts, or spikes should not be nearer the end of a timber member than six times their diameter ; and they should also be spaced that distance away from one another if in a straight line.

Where two pieces of timber, such as the rafter and tie beam of a plank truss are joined by direct spiking to one another, the nails and spikes should be staggered.

Where straps and bolts are used to connect the members of a truss the bearing of the bolt on the timber (or of the strap if there is no bolt) is the determining factor in the strength of the joint. In such cases to avoid the use of abnormal sizes lining tubes or shoe plates may be sometimes useful to increase the bearing surface.

The strap must be of thick enough metal not to fail by bearing on the bolt, as well as of sufficient section to take the direct tensile stress at its weakest section, *i.e.* :—where the bolt passes through it.

The use of the table will enable any of the joints in a timber roof to be designed directly from the stress diagram for the roof without further calculation, as it is readily seen by inspection what size of bolts and straps are required.

Riveted Joints.

In modern steel roof work riveted joints are almost exclusively used, and the work is so accurately set out that screw couplings or gib and cotter pins for tightening up are practically obsolete.

The calculation of the joints is therefore limited to seeing that there are sufficient rivets to take the stress tending to shear them, and that the plates are not subjected to undue bearing stress.

The following table extracted from Dorman, Long & Co.'s Pocket Companion is a very convenient one to use :—

Double shear is calculated at $1\frac{3}{4}$ times single shear and the shearing and bearing stresses reduced to 4 and 7 tons per square inch respectively to give extra safety in the joints.

TABLE XI.

SHEARING AND BEARING VALUE OF RIVETS IN TONS.

Diam. of Rivet in inches.	Area in square inches.	Shearing Value at 4 tons per sq. inch.		Bearing Values at 7 tons per square inch. Thickness in inches of Plate passed through.									
		Single Shear.	Double Shear.	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{7}{8}$
[illegible]	·1104	·44	·77	·66	·82								
[illegible]	·1963	·79	1·37	·87	1·09	1·31	1·53						
[illegible]	·3068	1·23	2·15	1·09	1·37	1·64	1·91	2·19					
[illegible]	·4418	1·77	3·09	1·31	1·64	1·97	2·30	2·62	2·95	3·28			
[illegible]	·6013	2·41	4·21	1·53	1·91	2·30	2·68	3·06	3·45	3·83	4·21		
1	·7854	3·14	5·50	1·75	2·19	2·62	3·06	3·50	3·94	4·37	4·81	5·25	6·12

In designing the joints and using the above table the following points should be taken into consideration :—

The pitch of rivets cannot be less than 2 × diameter or there is not room to form the head ; 3 × diameter is a better limit to adopt, and the use of $\frac{3}{4}$″ rivets at 3″ pitch is common practice in roof work.

Rivets must not be nearer the edge of the plate than $1\frac{1}{2}$ × diameter to centre of rivet hole ; 2 × diameter is better if reasonably practicable.

Rivets should not be more than 12 times the thickness of a single plate apart or rust is liable to get in between the plates.

A rivet should not be used through a greater total thickness of plates or bars than four times its own diameter.

Rivets should never be arranged to take direct tension ; joints should be designed with cover plates and straps or hangers to transmit the stress between members so that the rivets are only in shear.

Every joint should have at least two rivets, as it is not sound practice to rely on one only though it may be sufficient to transmit the stress.

It is customary to consider that riveting fills the holes so well that the rivet holes need not be deducted from the area of the cross section of a compression bar ; in the case of tension bars one or more rivet holes must be deducted according to the arrangement of the rivets which is adopted. In either case it is important to group the rivets symmetrically about the mean fibre.

III.—TIMBER ROOF CALCULATIONS.

General Notes on Design.

In roof design the simplest roof is the best, and the smaller the unsupported span that can be conveniently arranged, the lighter and cheaper the roof will be. In the majority of ordinary houses partition walls may be used to support purlins, and save the introduction of roof trusses as is necessary in long or wide buildings where no internal support is available.

The pitch given to the roof depends on the climatic conditions, *e.g.*, where rain and snow are prevalent a steep pitch is necessary ; but in those tropical climates where there is little rain a flat roof is suitable, and is more easily made thick to resist heat. The pitch also depends on the nature of the roof covering to be used, and whether it is intended to make use of the roof space for attics.

In ordinary house design for this climate a steep pitched roof gives more room in the attic storey, throws off the rain more readily, and has a better appearance from the outside.

In a building such as a Barrack Block where no use is made of the roof space the pitch may be lower with advantage, since the area of the roof to be covered is lessened, and less resistance to the wind is offered.

On the other hand it will be seen in drawing the stress diagrams of roofs that the stress in the tie bar and principal rafter of a truss is much increased by too flat a pitch, so that the economy in reduction of area is discounted by the heavier timber needed in the trusses. The pitch therefore should not be less than that given in Tables I. and II., and may often be advantageously somewhat greater.

Whatever slope is adopted should be used throughout the building, as the effect of different roof slopes on the same building is unsightly.

Horizontal valley gutters, and parapets behind which snow and leaves are likely to lodge, should be avoided, and a good eaves projection given to shelter the walls and carry the drip well clear of the building.

As to the design of the roof supports considerable judgment and experience are required to arrange the roof timbers to the best advantage.

Several recognized forms of roof trusses are used to suit different spans, and great variety of design is possible especially where conditions differ from the normal.

Some of the more usual forms of roof are given below, and by taking off the stresses and applying the data for calculation given in the

previous note suitable scantlings for any span and roof covering can be arrived at.

Battens or Boarding.

Where the size dictated by practical requirements is not obviously stronger than is actually necessary, the battens or boarding laid under the roof covering may be taken as beams supported on the rafters and uniformly loaded.

For Baltic fir the formulæ are (see Table V.) :—

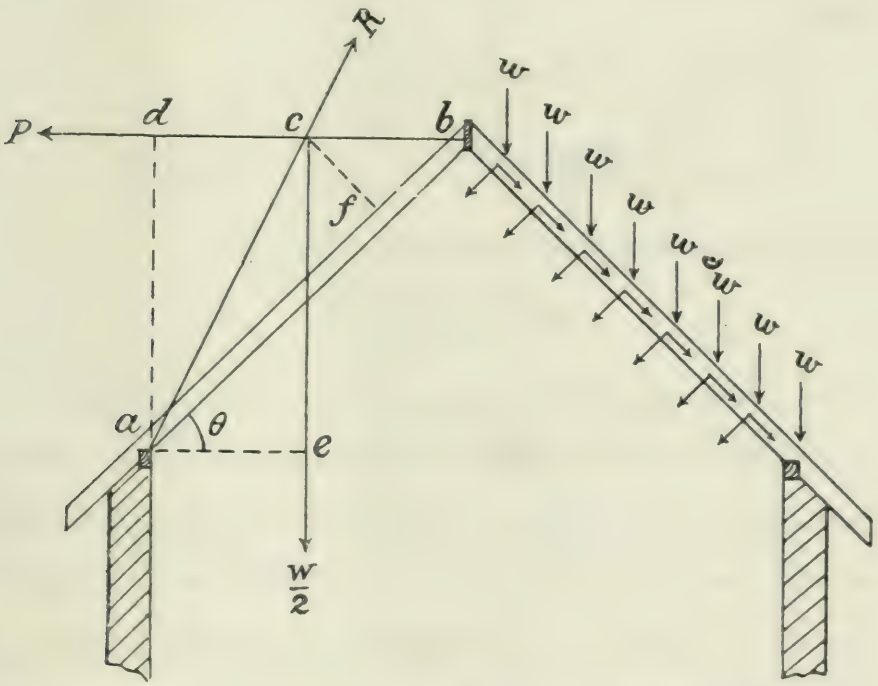
For strength $bd^2 = \frac{WL}{133}$

For deflection $bd^3 = \frac{WL}{133}$

The strength formula being used for the timber carrying corrugated iron or Eternit roofs where some deflection will do no harm to the roof covering, and the deflection formula for tiles or slates under which stiffer timbering is needed.

Couple Roof.

The Couple Roof (*Fig.* 1), or inclined rafter roof, is the simplest form of roof for spans not exceeding 12′ where brick or other substantial walls are available to take the outward thrust of the rafters.



Total Load on One Pair of Rafters = W.

FIG. 1.—*Couple Roof for Spans up to* 12′.

The rafters are notched over the wall plates and butt against the ridge board.

There is no transverse stress in the ridge board, which merely affords a convenient fixing to spike the upper ends of the rafters to; the pressure is transmitted down the rafters and met by an inclined resistance afforded by the wall plate.

The rafters are under combined stress, the component of the weight resolved normal to the roof slope tending to break the rafter across whilst the component resolved along the rafter is a direct thrust.

It is thus seen that the rafters of a pitched roof in all cases have to exert something of a strut action, and this should not be lost sight of even if for convenience they are often calculated merely as supported beams. In the extreme case of the rafters of a church spire they are almost direct struts with very little beam action at all.

To find the scantling that is suitable it is necessary to calculate what is the maximum fibre stress due to the transverse loading of the beam, and adding that to the fibre stress due to the direct stress, see that the combined stress does not produce a greater total stress on the fibres than is permitted by the table of safe working loads for struts (Table VIII.).

First to take the stress due to the bending moment we may treat the rafter as a supported beam uniformly loaded.

Then since the load causing the transverse stress is $\frac{W}{2}\cos\theta$ where $\frac{W}{2}=$ the load on one rafter and $\theta=$ the angle of inclination of the roof in this case

$$M_{ff}=\frac{W\cos\theta\times l}{2\times 8}$$

and

$$M_r=\frac{rbd^2}{6}.$$

Hence the fibre stress due to transverse load is

$$r=\frac{W\cos\theta\times l\times 6}{2\times 8\times bd^2}.$$

Next from the diagram of the stresses shown on the left-hand side of the roof we see that if ce represents the value of $\frac{W}{2}$ to any scale, then ea will represent P and ac will represent R in value and direction.

We also see that R may be resolved into $ad=ce=\frac{W}{2}=$ the downward pressure on wall and $ae=P=$the horizontal outward thrust against wall.

Further we see that the direct thrust in the rafter is made up of $af=$the resolved portion of R along the rafter and $bf=$the resolved portion of P along the rafter.

The direct thrust along the rafter is therefore represented by ab and $ab=\frac{W}{2}\operatorname{cosec}\theta$.

Hence the fibre stress due to direct thrust is

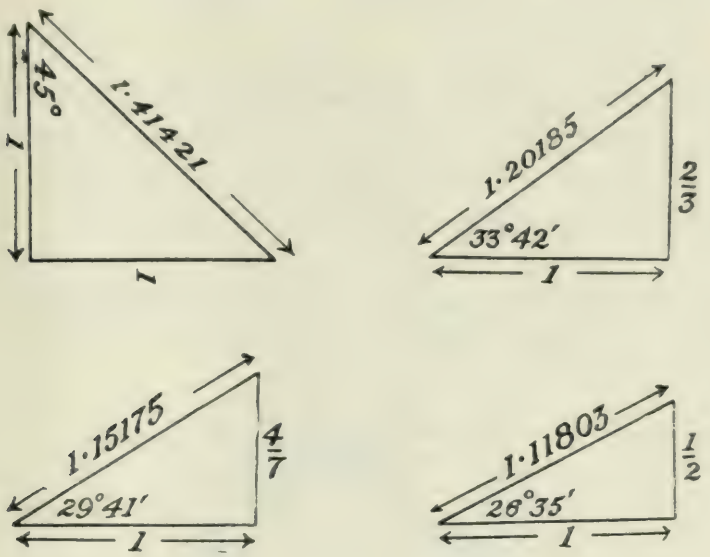
$$r_c=\frac{W\operatorname{cosec}\theta}{2\times bd}.$$

Thus the total fibre stress due to the combined stress is

$$r + r_c = \frac{W \cos\theta \times l \times 6}{2 \times 8b \times d^2} + \frac{W \operatorname{cosec}\theta}{2 \times bd}.$$

From this the scantling of the rafters suitable for any span and load can readily be worked out.*

It will be convenient here to show the lengths of rafters on the slope, and the value of the trigonometrical functions for the ordinary slopes used. Lengths of Slopes.



Slope.	Angle.	Sine.	Cosine.	Secant.	Cosecan
1 in 1	45° 0′	·70711	·70711	1·41421	1·41421
2 in 3	33° 42′	·55470	·83205	1·20185	1·80277
4 in 7	29° 41′	·49614	·86824	1·15175	2·01556
1 in 2	26° 35′	·44721	·89441	1·11803	2·23607

EXAMPLE I.—A couple roof is to be constructed over an out-building of span 10′ to carry a slated roof at a slope of 45°. The rafters are spaced at 1′ 6″ centre to centre. Find the size of rafters necessary. Timber, Baltic fir.

* Practically however as this calculation is a little laborious it will be sufficient in most cases to calculate common rafters by the simple formulæ $bd^2 = \frac{WL}{133}$ or $bd^3 = \frac{WL^2}{133}$ that is as supported beams, taking the full vertical load (Table III.) as if it were normal to the roof, and thus allowing some compensation for the fact that the rafter is partly acting as a beam and partly as a strut.

The load on the roof may be taken at 40 lbs. per square foot (Table III.).

The length of the rafter on the slope $= 5 \times 1{\cdot}41421 = 7'$ nearly.

The load on one rafter is therefore $7 \times 40 \times 1\frac{1}{2} = 420$ lbs.

The transverse load is then 420 lbs. $\times \cos\theta = 297$ lbs.

$$M_{ff} = \frac{297 \times 7 \times 12}{8} \text{ and } M_r = \frac{rbd^2}{6}.$$

$$\therefore \quad r = \frac{297 \times 7 \times 12 \times 6}{bd^2 \times 8} = \frac{18711}{bd^2}.$$

The direct stress is $\frac{W}{2}$ cosec $\theta = 594$ lbs.

$$\therefore \quad r_c = \frac{594}{bd}.$$

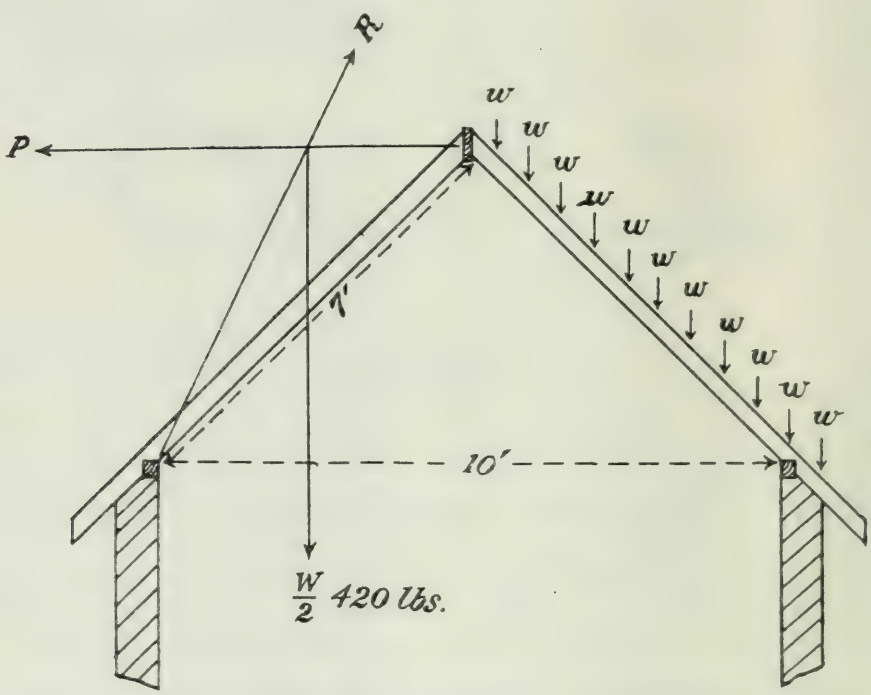


FIG. 2.

Assume a size of rafter which appears suitable.

Try $4''$ $\times 2''$ then $r + r_c = 585 + 74 = 659$ and $l : h :: 21 : 1$.

Try $4\frac{1}{2}'' \times 2''$ then $r + r_c = 462 + 66 = 528$ and $l : h :: 19 : 1$.

Try $5''$ $\times 2''$ then $r + r_c = 374 + 59 = 433$ and $l : h :: 17 : 1$.

(Note that for the least dimension the depth is taken, since the rafter is well fixed by the battens or boarding against any lateral movement).

Referring to Table VIII. we see that a rafter $4\frac{1}{2}'' \times 2''$ is suitable.

The alternative simpler method of working out this rafter as a supported beam gives the same result, thus :—

$$bd^3 = \frac{WL^2}{133}$$

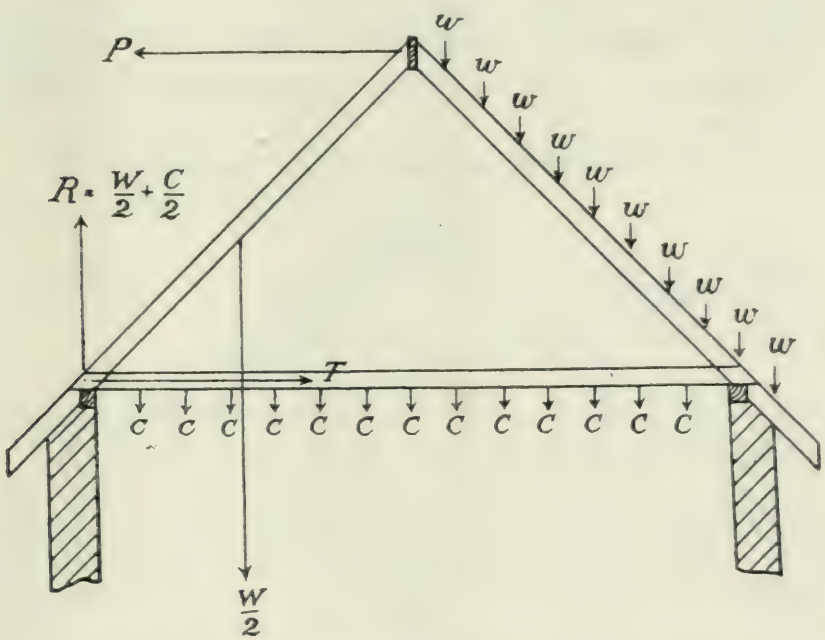
$$= \frac{420 \times 7 \times 7}{133}$$

$$= 155.$$

By Table VII. $4\frac{1}{2}'' \times 2''$ is suitable.

The Couple Close Roof (*Fig.* 3) or inclined rafter roof with tie beam is designed to prevent the tendency of the couple roof to thrust out the walls of the building by tying together the feet of the rafters either by an iron tie rod or a wooden beam, the latter being more suitable where a ceiling is required. Couple Close Roof.

It may be used for spans up to 18′ and is suitable for hut buildings with timber framed walls as well as on brick walls.

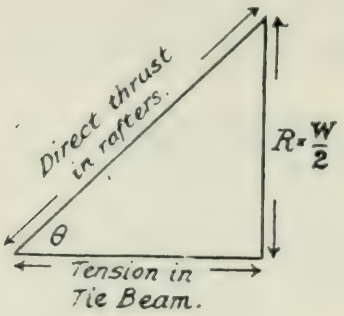


Total Load on One Truss = W. Ceiling Load = C.

FIG. 3.—*Couple Close Roof for Spans up to* 18′.

The roof covering is usually carried by boarding or battens laid on the rafters so that the rafters are under combined stress as before ; but the direct thrust at the foot of the rafter is taken up by the tie beam, so that the reaction from the wall is vertical and equivalent to the weight on half the truss $= \frac{W}{2}$ + half the ceiling load if there is

one $= \frac{C}{2}$. The stress diagram at the foot of the tie beam for the direct stresses is then :—



Direct Thrust in Rafter $= \frac{W}{2} \operatorname{cosec} \theta$

Tension in Tie $= \frac{W}{2} \cot \theta$

since the ceiling load does not affect the direct stresses.

The rafter may therefore be worked out exactly in the same way as for the couple roof. The tie beam is also subject to a combined stress if there is a ceiling attached to it, the maximum tension due to the direct stress must be added to that due to the transverse stress of the ceiling load and the weight of the beam itself, and should not exceed the safe limits given by Table IV.

EXAMPLE II.—A couple close roof is to be constructed to carry Eternit asbestos slates on a span of 15′ at a slope of 2 in 3. The rafters are spaced at 2′ 9″ c. to c. and a $\frac{5}{8}$″ matchboard ceiling is fixed to the tie beam.

Find the size of timbering necessary. Timber, Baltic fir.

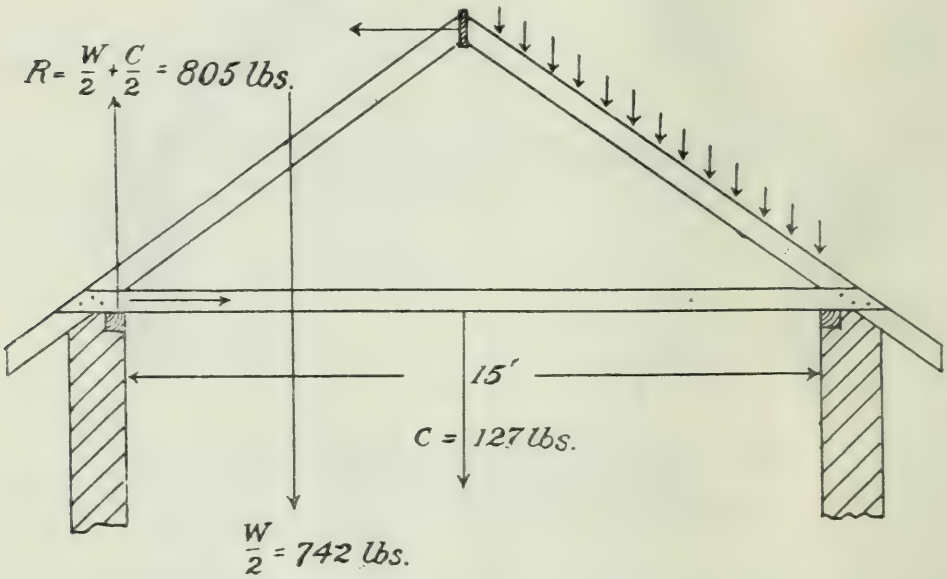


FIG. 4.

The load on the roof may be taken at 30 lbs. per square foot (Table III.).

The length of the rafter on the slope $= 7\frac{1}{2} \times 1{\cdot}20185 = 9'$ nearly.

The load on one rafter is therefore $9 \times 30 \times 2\frac{3}{4} = 742$ lbs.

The transverse load is then $742 \times \cos\theta = 617$ lbs.

$$\therefore \quad r = \frac{617 \times 9 \times 12 \times 6}{bd^2 \times 8} = \frac{49977}{bd^2}.$$

And the direct stress is $742 \times \text{cosec}\,\theta = 1378$ lbs.

$$\therefore \quad r_c = \frac{1378}{bd}.$$

Assume a size of rafter which appears suitable—

Try $6'' \times 3''$ then $r + r_c = 463 + 76 = 539$ and $l : h :: 18 : 1$.

Reference to Table VIII. shows that this size is suitable.

As the ceiling is a light one the tie beam may probably be a little lighter than the rafters.

Let us make it $6'' \times 2''$ and test for deflection.

The $\frac{5}{8}''$ matchboard ceiling weighs

$$2 \text{ lbs. per square foot} = 15 \times 2\tfrac{3}{4} \times 2 = 82 \text{ lbs.},$$

and the tie beam itself weighs

$$\frac{15 \times 6 \times 2 \times 36}{12 \times 12} = 45 \text{ lbs.}$$

The size of scantling to give a deflection of $\frac{1}{40}''$ per foot run is given by the formula

$$bd^3 = \frac{WL^2}{133} = \frac{127 \times 15 \times 15}{133} = 115 \text{ or } b = 2''\ d = 4''$$

will satisfy the requirement for deflection.

The direct stress in tie beam $= \frac{W}{2} \cot\theta = 742 \times \frac{3}{2} = 1{,}113$ lbs.

To allow for bolt holes and for cogging to wall plate the size $6'' \times 2''$ assumed is therefore suitable.

By inspection of Table XII. we see that a $\frac{1}{2}''$ bolt will be suitable to connect the tie beam and rafter; but as this must necessarily be placed rather near to the end of the tie beam it will be as well to add two 5" wire nails.

The joint should be made without weakening the timbers by halving them together.

The battens have a $2'\ 6''$ bearing and are laid at $9''$ centres for the usual diamond pattern Eternit roof. The size of ordinary slate battens, viz., $2'' \times \frac{3}{4}''$, will be suitable.

Collar Beam Roof.

The Collar Beam Roof is designed to give more headroom than the couple close roof whilst still relieving the walls of some of the outward pressure caused by the couple roof. The collar beam is usually placed from a quarter to halfway up the rafter according to the amount of headroom required. Unless the collar is kept quite low down this form of construction is an unsound one to employ except with walls of a substantial character.

The collars are usually fixed to each pair of rafters and carry the ceiling. Care should be taken to see that the rafter is not weakened by the form of joint adopted for fixing the collar. As the collar under certain conditions of stress may be brought into compression it is usually halved down on to the rafter to provide a shoulder bearing against the underside of the rafter; but it must not be reduced below the thickness necessary to give sufficient bearing to the bolt and nails securing the joint.

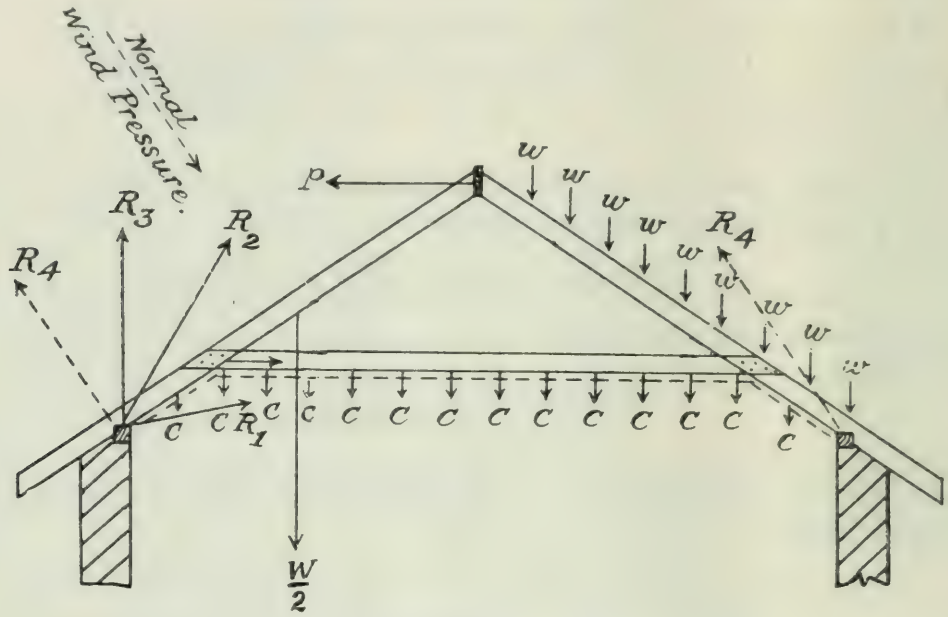


FIG. 5.—*Collar Beam Roof for Spans up to 18′.*

The stresses in the collar beam roof cannot be very easily calculated since the direction of the reaction at the walls will vary according to circumstances.

(1). With unyielding walls the reaction under permanent load will be approximately in the direction R_1; the collar beam is then in compression, and the rafter is stiffened by the addition of the collar.

(2). With the wall plate yielding slightly under pressure the reaction will be in the direction R_2, with no stress in the collar; the rafter is then in the same condition of stress as in the couple roof.

(3). With walls only capable of providing a vertical resistance R_3 the collar is in tension as in the couple close roof and the rafter has to bear additional cross strain at the joint with the collar.

(4). Again under wind pressure the reaction may be in the direction R_4 and the lower part of the rafter is in the condition of a cantilever.

The method of calculation to be adopted may then be as follows :—

(*a*). With substantial brick walls take the rafters as for the couple roof and make the collar beam of the same size. If the collar is

placed more than one-fourth up the rafter check the rafter as a cantilever under wind pressure, if a plaster ceiling is to be used check the collar for deflection.

(*b*). With light framed walls keep the collar down as low as possible and not in any case higher than one-fourth up the rafter, take the rafter as in the couple roof, but add for the additional transverse stress caused by the collar beam in tension, resolving the tensional stress normal to the rafter: tension in collar = pressure at ridge

$$= \frac{W}{2} \cot \theta.$$

EXAMPLE III. A collar beam roof is to carry ordinary tiles on a span of 15′ at a slope of 1 in 1. The rafters are spaced at 1′ 6″ c. to c. and the collars are halfway up the rafter. A plaster ceiling is fixed to the underside of the rafters and collar. The walls are brick 1′ 1½″ thick. Determine the size of the scantlings. Timber, Baltic fir.

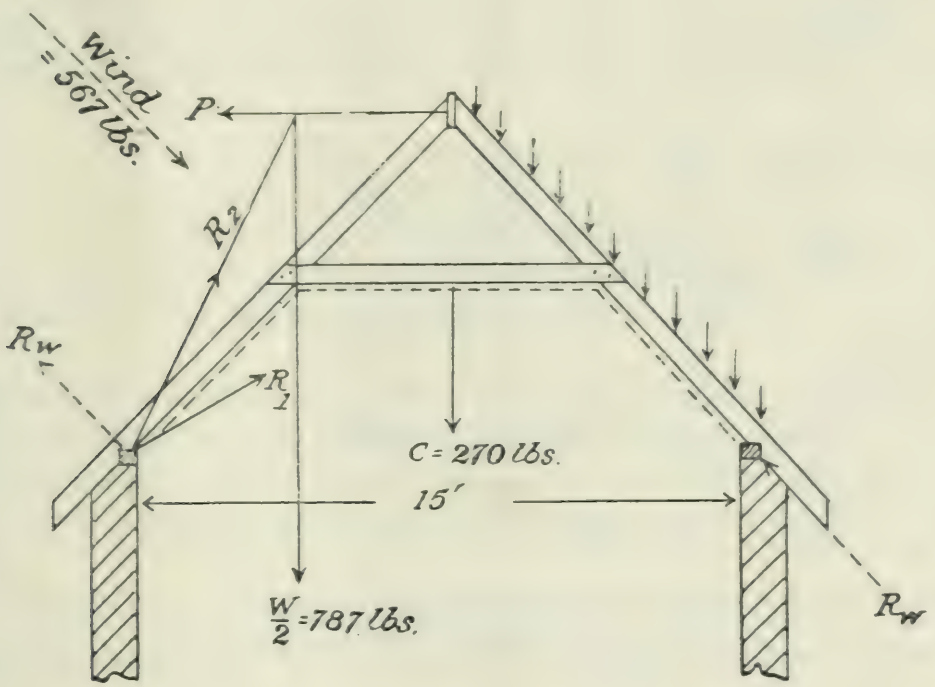


FIG. 6.

The load on the roof may be taken at 50 lbs. per square foot (Table III.). The length of the rafter on the slope

$$= 7\tfrac{1}{2} \times 1{\cdot}41421 = 10\tfrac{1}{2}' \text{ nearly.}$$

The load on one rafter is then

$$= 10\tfrac{1}{2} \times 50 \times 1\tfrac{1}{2} = 787 \text{ lbs.}$$

And the ceiling load on each bay

$$= 18 \times 1\tfrac{1}{2} \times 10 = 270 \text{ lbs.}$$

Taking the rafter as in a couple roof—that is on the supposition that the stress in the collar may at the worst be neutral and the reaction in the direction R_2—then the transverse load on the rafter is

$$787 \times \cos\theta = 557 \text{ lbs.}$$

due to the roof load and

$$5 \times 10 \times 1\tfrac{1}{2} = 75 \times \cos\theta = 53 \text{ lbs.}$$

due to the portion of the ceiling directly attached to the underside of the lower half of the rafter = total transverse load = 600 lbs.

$$\therefore \qquad r = \frac{600 \times 10\tfrac{1}{2} \times 12 \times 6}{bd^2 \times 8} = \frac{56700}{bd^2}$$

and the direct stress is $\frac{W}{2}$ cosec $\theta = 1{,}112$ lbs.

$$\therefore \qquad r_c = \frac{1112}{bd}.$$

Now the collar may very reasonably be considered to assist the rafter against buckling so that the least unsupported length may be taken at 5′, though we do not count on the collar against transverse stress.

Assume a size of rafter which appears suitable.

Try $5'' \times 2\tfrac{1}{2}''$ then $r + r_c = 907 + 89 = 996$ and $l : h :: 12 : 1$.

Try $5\tfrac{1}{2}'' \times 2\tfrac{3}{4}''$ then $r + r_c = 681 + 74 = 755$ and $l : h :: 11 : 1$.

Try $6'' \times 3''$ then $r + r_c = 525 + 62 = 587$ and $l : h :: 10 : 1$.

Reference to Table VIII. shows that a rafter $5\tfrac{1}{2}'' \times 2\tfrac{3}{4}''$ will be suitable.

Checking the lower portion of the rafter as a cantilever under wind pressure we may consider that as the rafter is continuous over the joint with the collar the lower portion is in the condition of a cantilever fixed at one end and loaded at the other with a load $= R_w = 283$ lbs.

The bending moment about the joint is $283 \times 5 \times 12 = 16{,}980''$ lbs., and the moment of resistance $= \frac{rbd^2}{6} = \frac{1200 \times 83{\cdot}2}{6} = 16{,}640''$ lbs. which is not quite sufficient.

As the rafter must necessarily be slightly weakened by the spiking of the collar to it, though it is not cut away at all, it will be as well to make it $6'' \times 3''$.

The collars may be of the same size and may be cut down 1″ on to the rafter to provide a shoulder against the underside of the rafter; they will then be strong enough to take any compression which may come upon them, and will evidently be amply stiff enough to carry the plaster ceiling.

The rafters and collars may therefore be all of $6'' \times 3''$ scantling if spaced at 1′ 6″ centre to centre.

When used as a truss to carry purlins it is sounder construction to tie in the feet of the rafters either with iron tie rods or wooden tie beams as shown in the two examples below. Collar Truss.

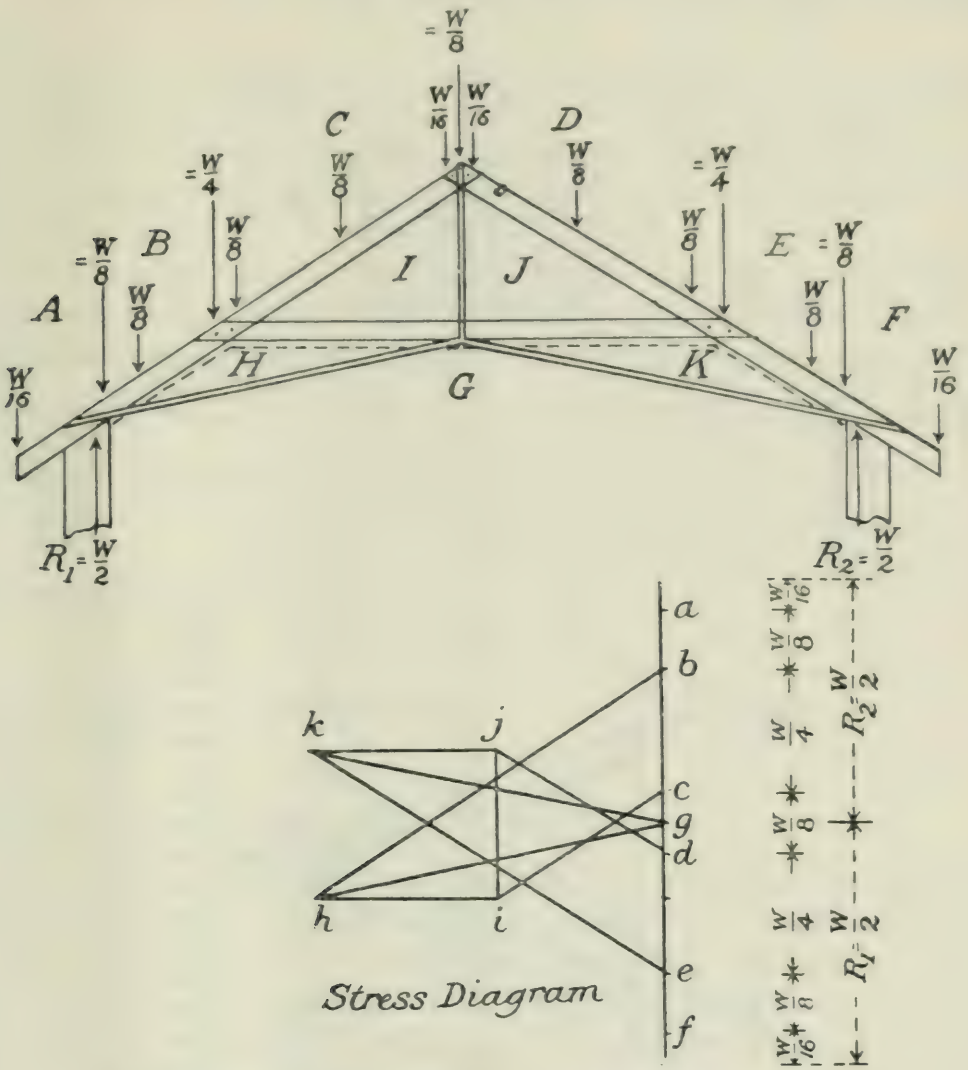


FIG. 7.—*Collar Beam Truss with Ties for Spans up to 24'.*

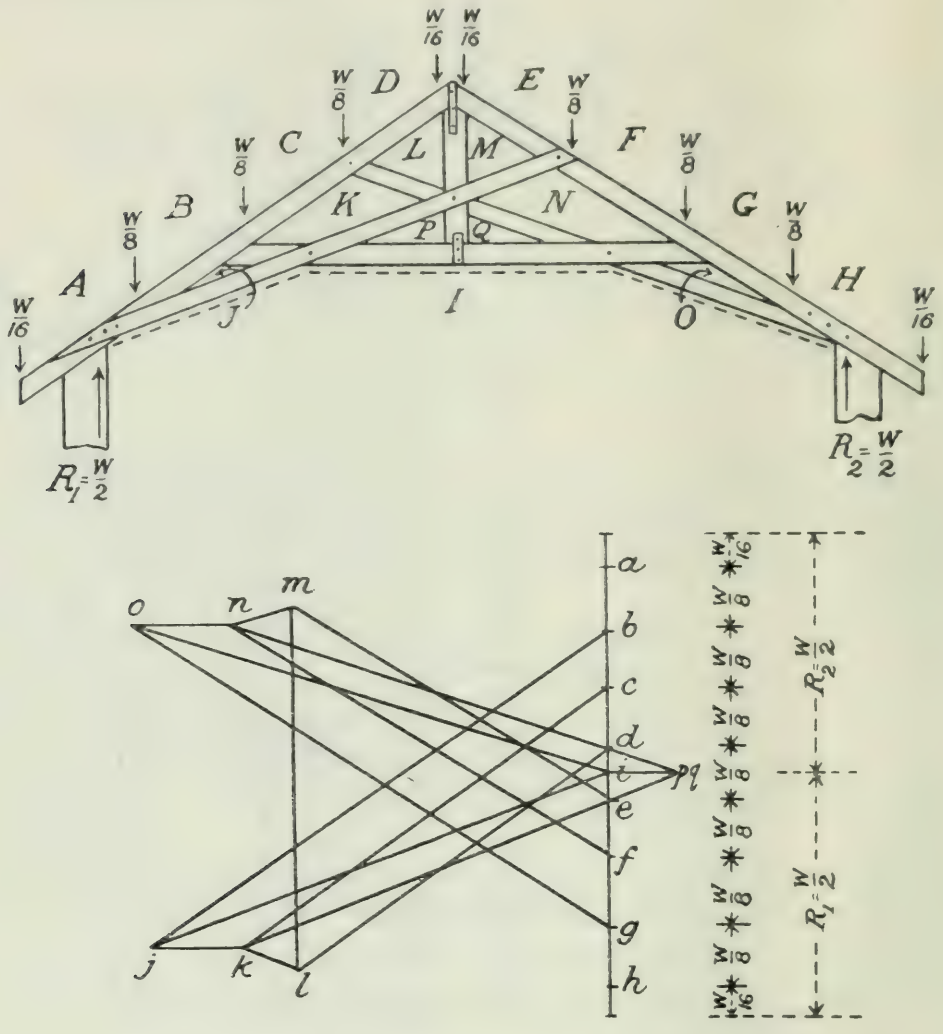
In this form of roof a complete truss is formed and if the purlin loads are replaced by equivalent load figures on the joints of the frame a stress diagram can be drawn and the direct stresses in the members ascertained.

The rafter is under combined stress owing to the position of the purlins, therefore the stress due to transverse loading must be added to the direct stress to find the size of scantling suitable.

A practical disadvantage of this form of roof is that the tie rods show below the ceiling, and are liable in barrack rooms to be pulled out of shape by unfair usage.

Another pattern of collar truss which has been used for barrack huts is the scissors truss (*Fig.* 8), this has the advantage of giving a coved ceiling with no exposed tie bars, and a good air space between the ceiling and roof covering. On the other hand it must be noted that Scissors Truss.

the acute angles at which the scissors beam meets the rafter causes considerable stress in these two principal members of the truss, and the joint between the two needs to be well secured. The timbers are also liable to warp if made too thin.



Stress Diagram.

FIG. 8.—*Scissors Truss for Spans up to 24′.*

As the rafter is supported at two points transverse stress is very greatly reduced, and the direct stresses can be taken off the stress diagram, from which it is practically only necessary to check the sizes of the members *bj* and *ji* since the truss will for convenience of construction be of the same size of scantling throughout.

EXAMPLE IV.—A hut barrack roof of 20′ span is to be constructed to carry galvanized corrugated iron laid on purlins. A $\frac{3}{4}''$ matchboard ceiling is to be fixed to the underside of the trusses which are to be of the scissors type. The trusses are at 5′ centres. Find the size of timbering necessary. Pitch 1 in 3. Timber, Baltic fir.

Draw a diagram of the roof as in *Fig.* 8.

Then with a roof slope of 2 in 3 the length of the rafter on the slope will be $10 \times 1{\cdot}20185 = 12'$ nearly, and to allow for the thickness of walls $= 6''$ and eaves projection $= 6''$ the overall length of the corrugated iron will be $13'$, so that $2 \times 7'$ sheets will be suitable, allowing $1'$ of lap.

One purlin is required at each side of the ridging to fix the upper end of the sheets and the ridge capping, one purlin is required at the eaves, also one under the lap of the sheets, and one purlin at the centre of each sheet; so that in all there will be five purlins at each side of the roof.

The load may then be divided as shown and the stress diagram drawn to a suitable scale (a good deal larger than shown in *Fig.* 8).

By measurement from the diagram we find that if the total load on one truss (including $\frac{W}{16}$ at the eaves which does not come into the diagram) at 20 lbs. per foot super.

$$= 13 \times 2 \times 5 \times 20 = 2{,}600 \text{ lbs.} = W,$$

then greatest stress in rafter

$$= bj = 3{,}000 \text{ lbs.},$$

and greatest stress in scissor beam

$$= ji = 2{,}600 \text{ lbs.},$$

and stress in king rod $= lm = 2{,}000$ lbs.

As the ceiling is a light matchboard one and the load taken is ample for corrugated iron the ceiling load may be disregarded.

Now the rafter is $12'$ in length and is supported at two intermediate points, so that its greatest unsupported length is $4'$; also it is assisted against lateral buckling by the purlins which are spaced $3'\ 3''$ apart. The purlins come so nearly over the points of support that transverse stress in the rafter is practically eliminated.

Let us assume a scantling of $6'' \times 1\frac{1}{2}''$.

Then $l : h :: 3'\ 3'' : 1\frac{1}{2}'' = 26 : 1$ so that the safe stress (Table VIII.) is 366 lbs. per square inch, or the rafter $6'' \times 1\frac{1}{2}''$ can take $9 \times 366 = 3{,}294$ lbs.

A scantling of $6'' \times 1\frac{1}{2}''$ is therefore suitable for the rafter, and it is evidently ample for all the other members.

The truss may therefore be all made up of $6'' \times 1\frac{1}{2}''$ timber.

The construction is as follows:—The rafters are laid down and halved together at the head; the collar beam is nailed to the underside of the rafter one third up the rafter, and a small piece of the same scantling is fixed as a king post and secured with hoop-iron straps bent over the head, and round the collar; the scissors beams are then laid one on each side of the truss, and the whole bolted together.

For the joint at the foot of the rafter with the scissors beam we see from Table XII. that with $1\frac{1}{2}''$ timber we require three $\frac{1}{2}''$ bolts to give sufficient bearing value, and these must be kept not less than $3''$ from the end of the scissors beam and $3''$ apart.

For the other joints a single $\frac{1}{2}''$ bolt will do.

For the purlins $3'' \times 2''$ laid on the flat is a convenient size for fixing corrugated iron and is strong enough for a $5'$ bearing at $3'$ $3''$ apart.

The $\frac{3}{4}''$ matchboard ceiling is nailed along the underside of the scissors beams and collar as dotted; a moulding under each truss screwed through to the beam will cover the end joints and improve the appearance of the ceiling.

Purlin Roof.

The Purlin and Rafter Roof (*Fig.* 9) is the type of roof adopted in ordinary house construction when the span exceeds that for which a collar beam roof can be safely used, and when partition walls, or corbels on chimney breasts, are available for the support of the purlins. It will be found economical to support the rafters at about 8 to 10′ intervals, and the size of purlins becomes excessive if they have to span more than about 12′.

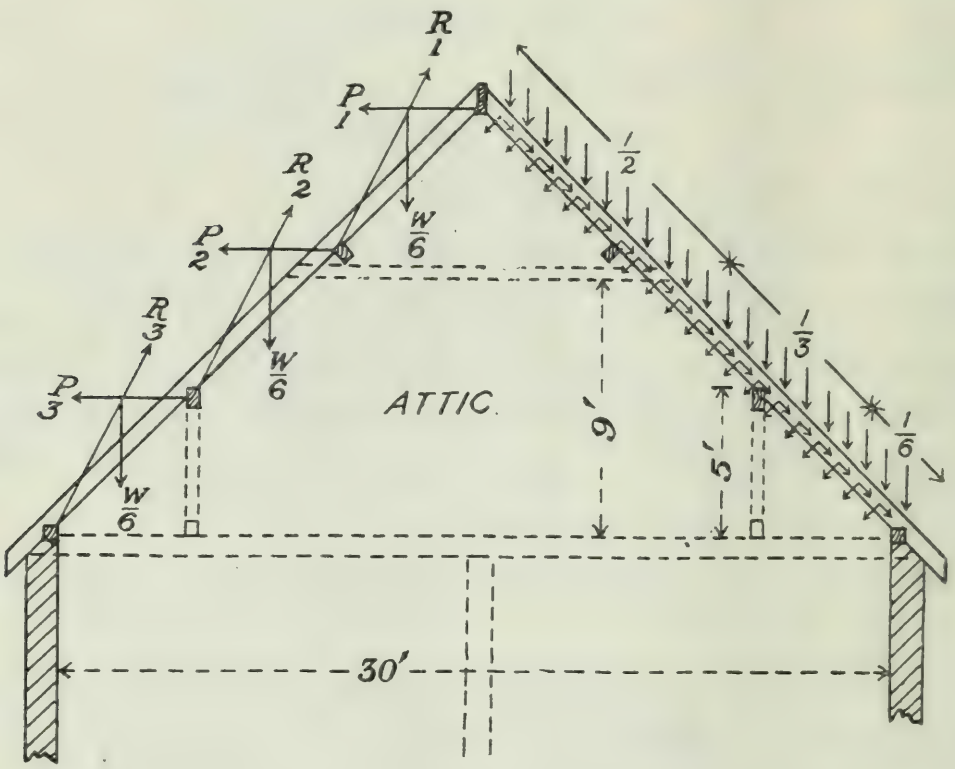


FIG. 9.—*Purlin Roof.*

There is no transverse stress in the ridge board.

The rafters are notched about $\frac{1}{2}''$ and spiked down to the purlins so that the downward thrust of each bay of the roof is taken up by the purlins or wall plate respectively.

Examining the distribution of the transverse load on the rafters as

it effects the loading of the purlins we see that with the usual equal spacing of purlins the upper purlin in *Fig.* 9 has to bear more than its share of the load, since the load coming on the ridge board is passed on down the rafter to the opposite purlin, and *vice versâ*.

Hence it appears that in considering the load taken by each purlin the portion of load causing transverse stress may be divided as shown in *Fig.* 10; and except where directly taken up by the template, or chock, or other lateral fixing of the purlin, the direct stress coming down the rafter as a thrust has also to be borne by the purlin, so that it will be practically correct to take the portion of the roof load coming on each purlin as a distributed load acting normal to the purlin and causing transverse stress, *i.e.*, the beam is for convenience worked out for the full surface load, so that the weakness of placing it diagonally instead of vertically is approximately compensated.

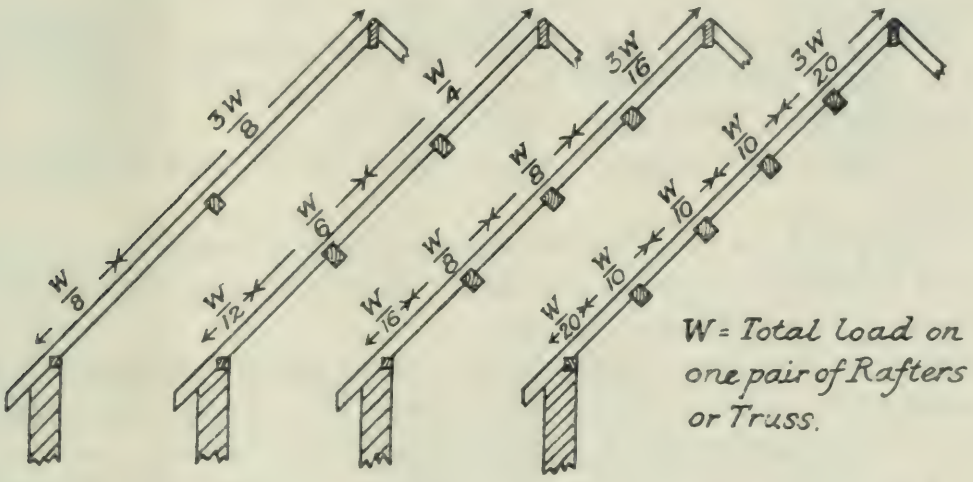


FIG. 10.—*Distribution of Load from Common Rafters on to Purlins.*

It is not practically worth while to try and space the purlins so as to get an equal load on each, for then the span of the rafters would be unequal.

It is also not worth while to introduce the theory that the rafter is a continuous beam, and may therefore in the end spans bring more load on to the purlin than on to the adjacent wall plate or ridge board; this theory is dependent on the supposition that the points of support remain exactly in one line, which in practice is not the case, since the purlins, and the truss, if any, supporting them, will deflect under load and throw the stress back on the wall plate and ridge board, which are comparatively rigid. The continuity of the rafter is also impaired by notching it down over the purlin, and in the case of a long slope it may be convenient to use two lengths of timber to form the common rafter, either in the original work or in the replacement of timber found defective.

It therefore appears best to consider the common rafters as

supported only at the purlins, and taking the simple division of loads as shown by *Fig.* 10 we may calculate the purlins by the ordinary formulæ for beams, taking the strength or deflection formula as required (Table V.).

The common rafters are calculated for direct and transverse stress as has been explained for couple roofs, and to save repetition of calculation tables of scantlings for various loads and spacing may conveniently be used.

The ridge board is made of convenient depth to fix the ends of the rafters against and $1\frac{1}{2}''$ in thickness.

Hip rafters receive the upper ends of the jack rafters, they may also be required to give support to the ends of purlins splayed and nailed to them ; their depth is therefore conveniently that from the upper edge of the rafters to the underside of the purlins, and $2''$ is the usual thickness. A special calculation for this scantling is seldom necessary.

Valley rafters are of the same depth and usually $3''$ in thickness will be found sufficient ; but they may be calculated readily enough by the method explained for rafters generally.

Wall plates are usually $4\frac{1}{2}'' \times 3''$ to work in with a course of brickwork.

Ceiling joists are usually $2''$ in thickness and $\frac{1}{2}''$ depth for each foot of span if spaced $1'$ apart in the clear ; their scantling may readily be checked by the deflection formula.

King Post Truss.

The King Post Truss is the form of truss usually adopted when it is required to support one purlin on each side of the roof, partition walls or other intermediate support not being available.

It is suitable up to $30'$ span ; but may be economically used up to $35'$ span if carrying a light roof covering and no ceiling.

Three examples will be given which will show the method of calculation under different circumstances.

EXAMPLE V.—A king post roof over a tropical barrack of $24'$ span is to be constructed to carry double Allahabad tiling laid on battens at $1'$ centre to centre laid on common rafters.

The spacing of trusses is $7'$ $6''$ centre to centre. Pitch 1 in 4. No ceiling. Timber, teak.

The load on the roof may be taken at 60 lbs. per square foot (Table III.).

The length of the rafter on the slope (taking $25'$ as the span for calculation)

$$= 12\tfrac{1}{2} \times 1{\cdot}11803 = 14' \text{ nearly.}$$

The total load on one truss

$$= W = 2 \times 14 \times 7\tfrac{1}{2} \times 60 = 12{,}600 \text{ lbs.}$$

From what has been explained of the loading of purlins under common rafters it will be clear that there is in this case no load on

the apex of the truss since the ridge board is not under transverse stress; though housed in to the top of the king post for convenience of fixing the roof it may be observed that the strut action of the common rafters on each side of the ridge board may when the truss deflects under load actually lift the ridge board from its housing.

The load distribution may then be taken as in *Fig.* 11.

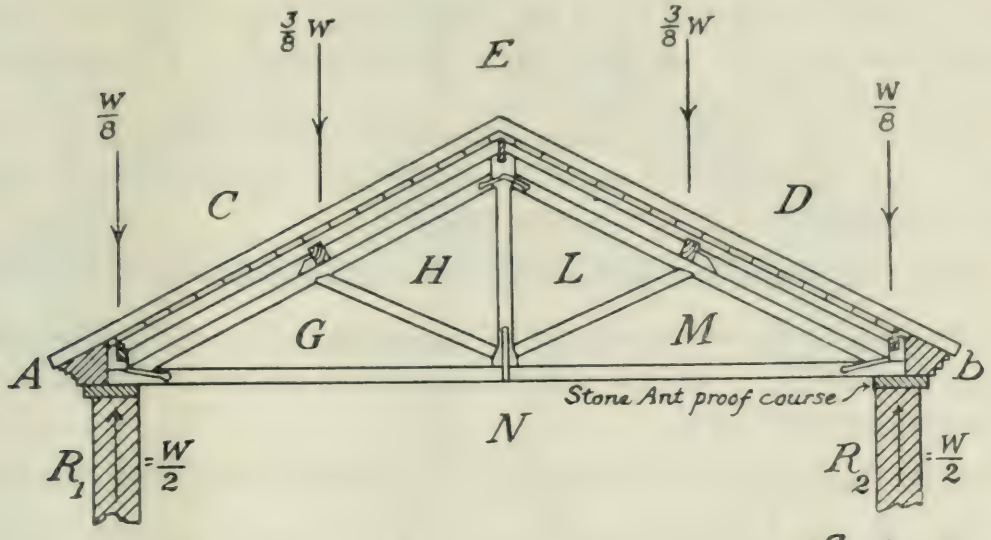
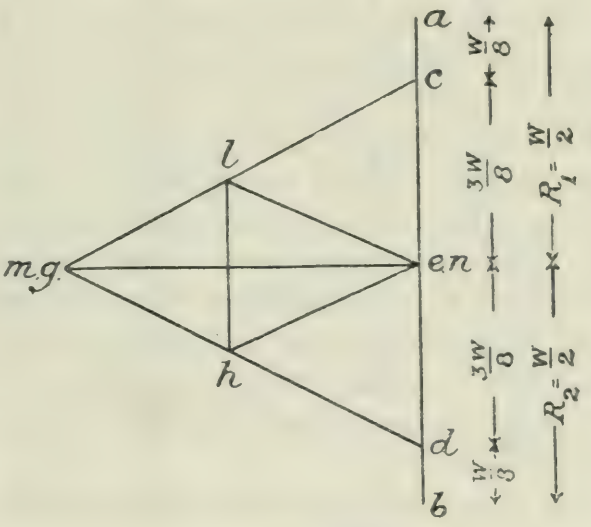


TABLE OF SCANTLINGS.

P.R.	$4\frac{1}{2}'' \times 4''$
Tie Beam	$6'' \times 4''$
King Post	$3'' \times 4''$
Struts	$3'' \times 4''$
Purlins	$7\frac{1}{4}'' \times 4\frac{3}{4}''$
Strap to P.R.	$2'' \times \frac{3}{8}''$



Stress Diagram.

FIG. 11.—*King Post Truss with Common Rafters on Purlins. No Ceiling Load.*

The calculation of battens and common rafters has already been explained and need not be repeated.

The purlins span 7′ 6″ and bear $\frac{3}{8} \times 12{,}600 = 4{,}725$ lbs.

By the deflection formula then

$$bd^3 = \frac{WL^2}{148} = \frac{4725 \times 7\frac{1}{2} \times 7\frac{1}{2}}{148} = 1{,}796.$$

From Table VII. we see that a size of $b = 4\frac{3}{4}''$ $d = 7\frac{1}{4}''$ will be suitable.

From the stress diagram (remembering that the slope of the roof is 1 in 2) we see that since $ce = \frac{3}{8}$ W = 4,725 lbs.

Stress in $cg = 4{,}725 \times 2 \times 1{\cdot}11803 = 10{,}565$ lbs. compression.
Stress in $gh = 4{,}725 \times 1{\cdot}11803 = 5{,}282$ lbs. compression.
Stress in $gn = 4{,}725 \times 2 = 9{,}450$ lbs. tension.
Stress in $hl = 4{,}725 = 4{,}725$ lbs. tension.

For principal rafter assume a least dimension of 4″ then $l : h :: 7' : 4'' = 21 : 1$. For teak $r_c = 1{,}340$ lbs. and from Table VIII. the safe reduced stress is 646 lbs. per square inch, so that 16·3″ is required. The principal rafter may be $4\frac{1}{2}'' \times 4''$.

For the strut assume a least dimension of 3″ then $l : h :: 7' : 3'' = 28 : 1$ the safe reduced stress is 440 lbs. per square inch, so that 11·8″ is required. The struts may be $3'' \times 4''$.

For the tie beam we require $\frac{9450}{1210} = 8$ square inches so that $4'' \times 2''$ will do; but to this we must add to the depth for the oblique mortice at the foot of the principal rafter and for the bolt hole for strap, so tie beam may be $6'' \times 4''$.

For the king post we require $\frac{4725}{1210} = 4$ square inches and allowing for the bolt hole the king post may be $4'' \times 3''$ made out of $4'' \times 6''$ to shape the shoulders at head and foot, or these shoulders may be pieced on as the timber used is expensive.

For the strap and bolt at foot of principal we see from Table XII. that only a 1″ bolt is required to take shearing stress=stress in principal, but that 2″ bearing on the tie beam is necessary, so we may use a $1\frac{1}{2}''$ bolt inside a 2″ lining tube; for the strap $\frac{3}{8}''$ thickness is required to give sufficient bearing on a $1\frac{1}{2}''$ bolt, and nearly 1″ of metal is required for the tensional stress in the strap, so that the double strap may be $2'' \times \frac{3}{8}''$ with forged eyes for the bolt to pass through.

For the straps at head and foot of king post $2'' \times \frac{1}{4}''$ will be suitable with $\frac{3}{4}''$ bolts.

The truss can now be designed and the detail of the joints worked out to large scale.

(For details of construction see *Military Works Handbook*.—India).

EXAMPLE VI.—A king post roof over a building of 24′ span is to be constructed to carry Welsh slates on $\frac{3}{4}''$ deal boarding laid on common rafters on purlins. Trusses are at 10′ centre to centre. Timber, Baltic fir.

Pitch 1 in 3. A plaster ceiling is carried on ceiling joists under the tie beam.

The load on the roof may be taken at 40 lbs. per square foot (Table III.).

The length of the rafter on the slope (taking 25′ as the span for calculation)

$$= 12\frac{1}{2} \times 1{\cdot}20185 = 15' \text{ nearly.}$$

The total load on one truss = W = 2 × 15 × 10 × 40 = 12,000 lbs.

As the slates are carried on common rafters over the purlins there is no apex load on the truss and the load distribution may be taken as shown on *Fig.* 12.

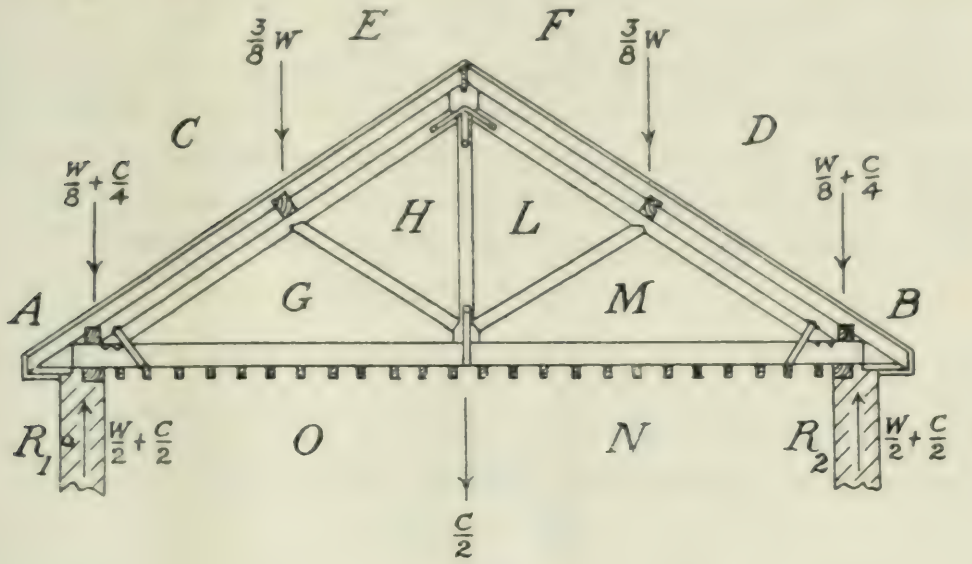


TABLE OF SCANTLINGS.

P.R.	$5\frac{1}{2}'' \times 4''$
Tie Beam	$9'' \times 4''$
King Post	$3'' \times 4''$
Struts	$3\frac{1}{2}'' \times 4''$
Purlins	$8\frac{1}{2}'' \times 5\frac{1}{2}''$
Strap to P.R.	$2'' \times \frac{3}{8}''$

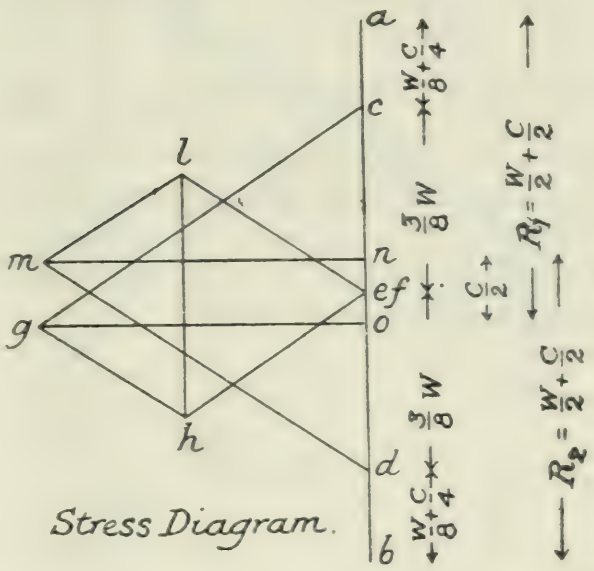


FIG. 12.—*King Post Truss with Common Rafters on Purlins. With Ceiling Load.*

The ceiling load = C = 24 × 10 × $12\frac{1}{2}$ = 3,000 lbs.

The tie beam is a continuous beam supported by the strap at the foot of the king post; but it is also partially fixed at the ends by the weight coming on the pole plate and by the strap holding the principal rafter. The ceiling load may therefore be taken as acting half at the foot of the king post and a quarter on each wall plate.

The calculation of boarding and common rafters has already been explained and need not be repeated.

The purlins span 10′ and bear $\frac{3}{8}$ × 12,000 = 4,500 lbs.

By the deflection formula then

$$bd^3 = \frac{WL^2}{133} = \frac{4500 \times 10 \times 10}{133} = 3375.$$

From Table VII. we see that a size of $b=5\frac{1}{2}''$ $d=8\frac{1}{2}''$ will be suitable.

From the stress diagram (remembering that the slope of the roof is 2 in 3) we see that since $co=\frac{3}{8}$ W $+\frac{1}{4}$ $c=5{,}225$ lbs.

Stress in $cg=co\times\frac{3}{2}\times 1{\cdot}20185=5{,}225\times\frac{3}{2}\times 1{\cdot}20185=9{,}405$ lbs. compression.

Stress in $gh=\frac{1}{2}\times ce\times\frac{3}{2}\times 1{\cdot}20185=\frac{1}{2}\times 4{,}500\times\frac{3}{2}\times 1{\cdot}20185=4{,}050$ lbs. compression.

Stress in $go=co\times\frac{3}{2}=5{,}225\times\frac{3}{2}=7{,}837$ lbs. tension.

Stress in $hl=ce+on=4{,}500+1{,}500=6{,}000$ lbs. tension.

For principal rafter assume a least dimension of 4″ then

$$l:h::7'\ 6'':4''=22\tfrac{1}{2}:1.$$

From Table VIII. the safe reduced stress is 437 lbs. per square inch so that 21·5 inches is required.

The principal rafter may be $5\frac{1}{2}''\times 4''$.

For the strut assume a least dimension of $3\frac{1}{2}''$ then

$$l:h::7'\ 6'':3\tfrac{1}{2}=26:1$$

the safe reduced stress is 366 lbs. per square inch so that 11″ is required. The strut may be $3\frac{1}{2}''\times 4''$.

For the tie beam we require $\frac{7837}{1400}=5\frac{1}{2}$ square inches for the direct tension, but this member is also under combined stress from the ceiling load, and must be stiff enough not to deflect more than $\frac{1}{400}''$ per foot run.

Taking the portion *go* as if supported only at each end and using the simple deflection formula

$$bd^3=\frac{WL^2}{133}$$

we have
$$bd^3=\frac{1500\times 10\times 10}{133}=1125$$

or
$$b=4''\ d=5\tfrac{3}{4}''.$$

Allowing for the oblique mortice at the foot of the principal and notch for strap we may make the tie beam $9''\times 4''$.

For the king post we require $\frac{6000}{1400}=3\frac{1}{2}$ square inches. Allowing for the bolt hole the king post may be $4''\times 3''$ made out of $4''\times 6''$ to shape the shoulders at head and foot.

For the strap at the foot of principal we see from Table XII. that if the strap bears direct on the underside of the notch on the tie beam a bearing of 2″ width is required to take a stress equal to that in the principal rafter, and nearly 1 square inch of metal is required for the tensional stress in the strap. To give sufficient metal to forge up the screwed ends of the strap it will be as well to make the strap at the foot of the principal $2''\times\frac{3}{8}''$.

For the straps at head and foot of the king post 2″ × ¼″ straps will do, with ⅝″ bolts, two bolts being required in each vertical strap to take the bearing stress equal to tension in the king post.

The truss can now be designed and the detail of joints worked out to large scale.

EXAMPLE VII.—A king post roof over a troop stable of 31′ span is to be constructed to carry a roof of ruberoid laid on 1″ deal boarding resting direct on purlins spaced 3′ apart. Trusses are at 11′ centre to centre. Timber, Baltic fir. Pitch 1 in 3.

There is no ceiling, but there are ventilating lanterns between each alternate pair of trusses.

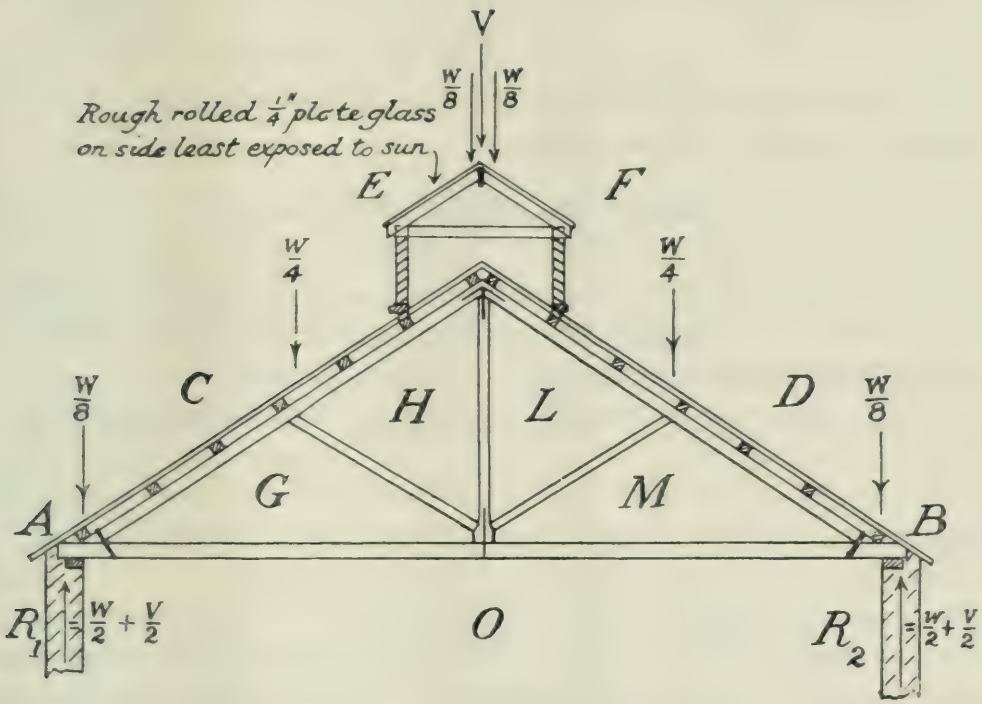


TABLE OF SCANTLINGS.

P.R.	6½″ × 3″
Tie Beam	6½″ × 3″
King Post	3½″ × 3″
Struts	3½″ × 3″
Purlins	4″ × 3½″
Strap to P.R.	2″ × ¼″

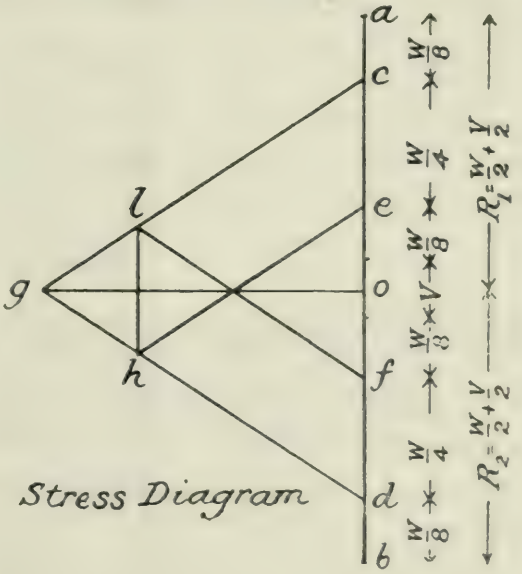


FIG. 13.—*King Post Truss with Purlins along the Principal Rafter. No Ceiling.*

The load on the roof may be taken at 20 lbs. per square foot (Table III.).

The length of the rafter on the slope (taking $32'$ as the span for calculation) $= 16 \times 1{\cdot}20185 = 19\frac{1}{4}'$ nearly.

The total load on one truss $= W = 2 \times 19\frac{1}{4} \times 11 \times 20 = 8{,}470$ lbs.

And the extra load due to the lanterns between alternate trusses over and above the 20 lbs. allowance = about 700 lbs.

As the roof covering is carried by purlins laid direct on the principal rafter the principal rafter is under transverse load as well as direct stress.

The roof load may therefore be divided as shown on *Fig.* 13, viz., $\frac{W}{4}$ at the apex and over each strut and $\frac{W}{8}$ over the wall plate.

The lantern load may also be taken at the apex.

The purlins span $11'$ and bear $3 \times 11 \times 20 = 660$ lbs.

By the strength formula then

$$bd^2 = \frac{WL}{133} = \frac{3 \times 11 \times 20 \times 11}{133} = 54.$$

From Table VI. we see that a size of $b = 3\frac{1}{2}''$ $d = 4''$ will be suitable.

From the stress diagram (remembering that the slope of the roof is 1 in 3) we see that since $co = \frac{3}{8}W + \frac{1}{2}V = 3{,}526$ lbs.

Stress in $cg = co \times \frac{3}{2} \times 1{\cdot}20185 = 3526 \times \frac{3}{2} \times 1{\cdot}20185 = 6{,}347$ lbs. compression.

Stress in $gh = \frac{1}{2} \times ce \times \frac{3}{2} \times 1{\cdot}20185 = \frac{1}{2} \times 2118 \times \frac{3}{2} \times 1{\cdot}20185 = 1{,}906$ lbs. compression.

Stress in $go = co \times \frac{3}{2} = 3526 \times \frac{3}{2} = 5{,}289$ lbs. tension.

Stress in $lh = ce = 2118 = 2118$ lbs. tension.

The principal rafter is also under transverse stress being loaded along its length by purlins at $3'$ intervals. This is practically equivalent to a uniformly distributed load on the portion

$$cg = \frac{W}{4} \cos \theta.$$

then
$$M_{ff} = \frac{W \cos \theta \times l}{4 \times 8}$$

and
$$M_r = \frac{rbd^2}{6}.$$

Hence the fibre stress due to transverse load is

$$r = \frac{W \cos \theta \times l \times 6}{4 \times 8 \times bd^2} = \frac{8470 \times {\cdot}83205 \times 77 \times 12 \times 6}{4 \times 8 \times 8 \times bd^2} = \frac{152621}{bd^2}.$$

And from the stress diagram the fibre stress due to the direct thrust, *i.e.*,

$$r_c = \frac{6347}{bd}.$$

Assume a size of rafter which appears suitable.

Try $6'' \times 3''$ then $r + r_c = 236 + 353 = 589$.

Try $6\frac{1}{2}'' \times 3''$ then $r + r_c = 185 + 326 = 511$.

Now to apply Table VIII. we may consider that the purlins give some lateral fixation so that $3'$ is the greatest unsupported length with regard to the breadth of the rafter, whilst $9'\ 6''$ is the greatest unsupported length with regard to its depth : the stress must be reduced to the lower figure of the two.

For $6'' \times 3''$ the $l : h :: 12 : 1$ laterally or $:: 19 : 1$ vertically.

For $6\frac{1}{2}'' \times 3''$ the $l : h :: 12 : 1$ laterally or $:: 18 : 1$ vertically.

From Table VIII. then we see that $6'' \times 3''$ is not quite strong enough and we may decide to use $6\frac{1}{2}'' \times 3''$ for the principal rafter.

For the tie beam as there is no ceiling the same dimension, viz., $6\frac{1}{2}'' \times 3''$ will be suitable.

For the struts assume a least dimension of $3''$ then

$$l : h :: 9'\ 6'' : 3'' = 38 : 1$$

the safe reduced stress is just under 200 lbs. per square inch so that about 10 square inches is required. The struts may be $3\frac{1}{2}'' \times 3''$.

The same dimensions, viz., $3\frac{1}{2}'' \times 3''$, will be suitable for the king post, but made of $6'' \times 3''$ to shape the shoulders.

From Table XII. we see that $2'' \times \frac{1}{4}''$ will be suitable for the strap at the foot of the principal rafter and for the king post with two $\frac{1}{2}''$ bolts in the king post or a $\frac{5}{8}''$ gib and cotter if used for the foot of king post.

The truss can now be designed and details worked out to a large scale.

The Queen Post Truss is the form of truss usually adopted when it is required to support two purlins on each side of the roof. Queen Post Truss.

It is suitable up to $45'$ span and it is seldom that a timber truss will be used for any greater span than this. The method of calculation under permanent load is similar to that adopted for the king post truss so that one example only will be given. It must however be noticed that under wind pressure or other unequal loading the centre panel of the frame is unbraced so that unforeseen stresses are brought on the joints. For this reason the queen post truss is generally made of more substantial timber than the calculations show to be actually necessary.

The modern tendency is to abandon large span timber trusses and to use steel for any span greater than that for which the king post truss will suffice. On the other hand when used without a ceiling with timbers wrought and stop chamfered the queen post truss is a good deal more pleasing in appearance.

EXAMPLE VIII.—A queen post roof over a building of $44'$ span is to be constructed to carry Welsh slates on $\frac{3}{4}''$ deal boarding laid on common rafters on purlins. Trusses are at $12'$ centre to centre.

Timber, Baltic fir. Pitch 1 in 4. A plaster ceiling is carried on ceiling joists under the tie beam.

The load on the roof may be taken at 40 lbs. per square foot (Table III.).

The length of the rafter on the slope (taking 45′ as the span for calculation) $= 22\frac{1}{2} \times 1{\cdot}11803 = 25'$ nearly.

The total load on one truss $= W = 2 \times 25 \times 12 \times 40 = 24{,}000$ lbs.

And the ceiling load $= C = 44 \times 12 \times 12\frac{1}{2} = 6{,}600$ lbs.

The loads may be divided as shown in *Fig.* 14.

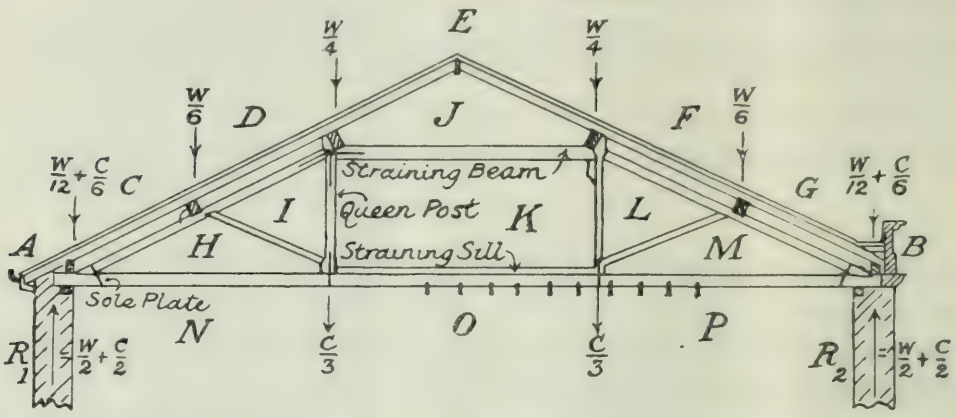
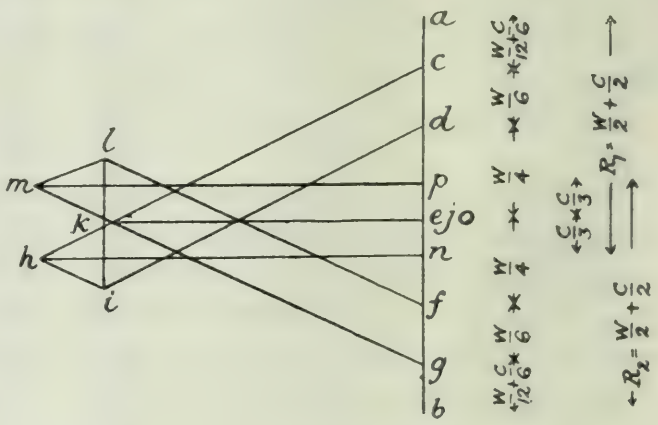


TABLE OF SCANTLINGS.

P.R.	8″ × 6″
Tie Beam	12″ × 6″
Queen Posts	4″ × 6″
Struts	4″ × 6″
Straining Beam	10″ × 6″
Purlins	10″ × $6\frac{1}{2}$″
Strap to P.R.	$2\frac{1}{2}$″ × $\frac{1}{2}$″



Stress Diagram.

FIG. 14.—*Queen Post Truss with Common Rafters on Purlins. With Ceiling Load.*

The purlins span 12′ and bear on the upper two purlins 6,000 lbs. By the deflection formula then :—

$$bd^3 = \frac{WL^2}{133} = \frac{6000 \times 12 \times 12}{133} = 6{,}480$$

so that

$$b = 6\tfrac{1}{2}''\ d = 10''$$

is needed, the two lower purlins might be a little lighter, but it is not advisable to change the section.

From the stress diagram we can obtain the direct stresses partly by computation and partly by measurement :—

Stress in $ch = 27{,}280$ lbs. compression.

Stress in $hi = 4{,}470$ lbs. compression.

Stress in $jk = 20{,}400$ lbs. compression.

Stress in $hn = 24{,}400$ lbs. tension.

Stress in $ik = 8{,}400$ lbs. tension.

For principal rafter assume a least dimension of 6″ then

$$l : h :: 8' : 6'' = 16 : 1.$$

From Table VIII. the safe reduced stress is 600 lbs. per square inch so that 48 square inches is required.

∴ the principal rafter may be $8'' \times 6''$.

For struts assume a least dimension of 4″ then

$$l : h :: 6' : 4'' = 18 : 1$$

and the safe reduced stress is 550 lbs. per square inch so that 8 square inches would do ; but it is advisable to make the struts amply strong to avoid distortion under wind pressure and eccentric loading.

∴ let us make the struts $4'' \times 6''$.

For straining beam take the least dimension at 6″ then

$$l : h :: 14' : 6'' = 28 : 1$$

and the safe reduced stress is 333 lbs. per square inch so that 60 square inches is required.

∴ the straining beam may be $10'' \times 6''$.

For tie beam for direct tension we require

$$\frac{24400}{1400} = 17\tfrac{1}{2} \text{ square inches.}$$

For deflection the length between supports is

$$\frac{44}{3} = 14'\ 8''$$

then

$$bd^3 = \frac{WL^2}{133} = \frac{2200 \times 44 \times 44}{133 \times 3 \times 3} = 3569 \text{ or } b = 6''\ d = 8\tfrac{1}{2}''.$$

To this we must add about $3\tfrac{1}{2}''$ for the oblique mortice at the foot of the principal and the notch for sole plate for strap.

∴ the tie beam may be $12'' \times 6''$.

For the queen post we require

$$\frac{8400}{1400} = 6 \text{ square inches.}$$

∴ to allow for bolt holes the queen posts may be $4'' \times 6''$.

For the strap at foot of principal we see that to take a stress = that in the principal rafter a bearing of $3\tfrac{1}{2}''$ on the timber is required so that a bolt through the tie beam is impracticable.

The best method will be to fix a sole plate under the tie beam giving not less than $6'' \times 3\frac{1}{2}''$ bearing and to use a double strap with another bearing plate above the rafter.

The strap requires nearly $2\frac{1}{2}''$ of metal so that $2\frac{1}{2}'' \times \frac{1}{2}''$ will be a suitable size for a double strap forged with screw bolts of equal strength at the ends.

The straps at the head and feet of the queen post may be $2'' \times \frac{3}{8}''$ with two $\frac{5}{8}''$ bolts at each bearing.

The truss can now be designed and the detail of the joints worked out to a large scale.

NOTE.—The diagrams of trusses have purposely been kept very small so as to read with the letterpress, as it is not possible on such a scale to properly show the detail of joints and fastenings. Readers who desire further information on these points are referred to the various lithographs, models, and diagrams in the Construction School, S.M.E., which show a great variety of such details.—[EDITOR.]

IV.—STEEL ROOF CALCULATIONS.

General Notes on Design.

Steel roof trusses are generally more economical than timber trusses for spans of 40′ and upwards; they are also used for smaller spans under special circumstances, *e.g.*, in tropical countries where heavy roof coverings have to be carried, especially where there is danger of timber being eaten by white ants; and in cases such as open drill sheds or railway stations where steel will suffer less than timber from exposure to the weather.

Steel is by reason of its manufacture a more reliable material than timber, and can therefore be safely used with a lower factor of safety, 4 is usually taken as the factor of safety for steel in roof work as against 5 for timber.

The members of a steel truss are made of the form best suited to take the stresses which theory shows they will be called on to bear, *e.g.*, principal rafters and struts may be of tee or angle steel, and ties of flat bars; the design should therefore be such as to bring only direct stresses to bear on each member; the principal rafter may, however, have to carry purlins spaced along its length, and occasionally the tie bar may have to carry a suspended ceiling.

Greater exactitude of calculation is therefore desirable than was adopted in the case of timber roofs; whereas in timber roofs of small span it was considered sufficient to take an allowance of dead load to cover either wind pressure or workmen's load it is necessary in steel roofs to work out the effect of wind pressure blowing on either side of the roof, since wind may have a serious effect on a large span roof, and might even cause reversal of stress in some cases, which would be fatal to light tension members.

Arrangements must also be made in all cases for expansion and contraction due to rise and fall of temperature, and this will further somewhat complicate the stresses.

Pitch of Roofs.

Steel roof trusses will seldom be used for buildings requiring a steep pitched roof, and in order to avoid long struts the tendency is to keep the roof slope as flat as other practical requirements will admit; the pitch of roofs with steel trusses may thus be taken at about the following figures:—

Rise, Span.	Slope.	Angle of Slope.	Length of Slope, Base=1.	Roof Covering.
$\frac{1}{3}$	1 in $1\frac{1}{2}$	33° 42′	1·20185	Slates in exposed situation.
$\frac{1}{4}$	1 in 2	26° 35′	1·11803	Slates in sheltered situation.
$\frac{1}{6}$	1 in 3	18° 25′	1·05409	Corrugated iron or ruberoid.

A camber is usually given in the tie bar to slightly further reduce the length of the struts, and to avoid the appearance of sagging which a perfectly horizontal bar would have ; the camber should not exceed $\frac{1}{40}$th or $\frac{1}{30}$th of the span or excessive strain would be set up both in tie and principal.

Form of Truss. It is found economical to design the form of truss so that the principal rafter obtains support at about every 8′ along its length. Economy of material is also obtained by keeping the compression members short, so that struts are best placed at right angles to the rafter or nearly so.

Three forms of truss design are in common use :—

(i.). The "Trussed Rafter" type shown in figures t_1, t_2, t_3, t_4, which is the most economical for a comparatively steep pitch of roof.

(ii.). The "Right Angle Strut" type shown in figures r_2, r_3, r_4, best suited for a flat pitch.

(iii.). The "King and Queen Rod" type with vertical suspension rods which gives a roof of better appearance and more convenient for hipping back ; these are shown in figures k_1, k_2, k_3, k_4.

Spacing of Trusses. The spacing of the trusses may be affected by exigencies of plan, such as position of columns, window and door openings ; but in most cases is solely a matter of practical economy.

Wider spacing saves in number of trusses but increases the load on each and necessitates heavier purlins ; rolled steel joists or in large roofs trussed girders may be economically used in place of wooden purlins.

For a light roof covering such as corrugated iron the spacing may be relatively wide, whilst for a heavy roof covering, such as double Allahabad tiling, close spacing is necessary.

Generally speaking, for an ordinary roof load such as slates on boarding $\frac{1}{3}$rd to $\frac{1}{5}$th the span are the usual limits of economy, *e.g.*—

For 30′ span take $\frac{1}{3}$ span and space 10′ apart.

For 50′ span take $\frac{1}{4}$ span and space $12\frac{1}{2}$′ apart.

For 70′ span take $\frac{1}{5}$ span and space 14′ apart.

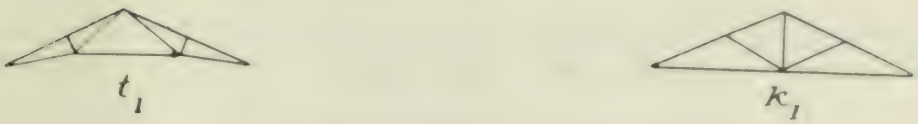
For heavy loads space rather closer, and for light loads rather wider than for slates.

In important cases it will be worth while to work out the weight and cost of trusses and purlins for a few possible spacings so as to actually prove which is the most economical.

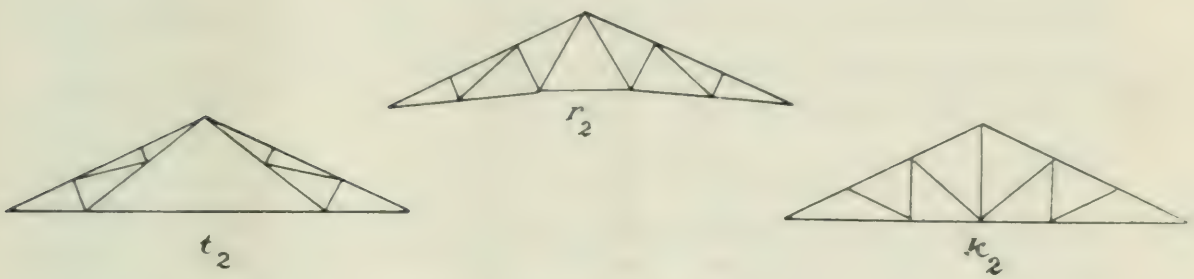
Weight of Trusses. An approximation to the weight of the truss has to be made to get at the total dead load for calculation.

The following rough rules will be useful though admittedly

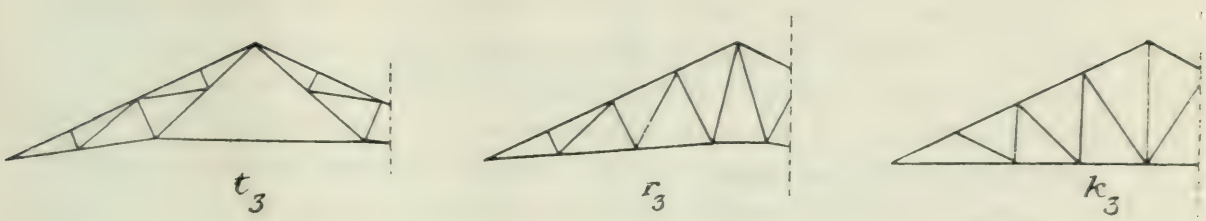
Span 20 to 30 feet (one strut)



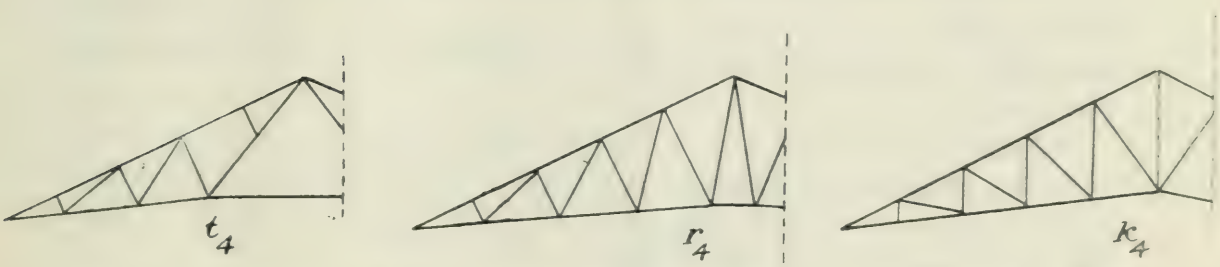
Span 30 to 45 feet (two struts)



Span 45 to 60 feet (three struts)



Span 60 to 75 feet (four struts)



Trussed rafter type. *Right angular strut type.* *King rod type*

FIG. 15.—*Types of Steel Trusses.*

inaccurate as they take no account of difference in loading or spacing of trusses :—

$$w = \frac{\text{weight in lbs. per sq. ft.}}{\text{of covered area}} = \frac{\text{span in feet}}{25} + 2,$$

or w = weight in lbs. per sq. ft. = ·08 × span in feet,
or another rule taking spacing into account is :—

$$W = \frac{\text{total weight of}}{\text{truss in lbs.}} = \tfrac{3}{4} \times \frac{\text{spacing}}{\text{in feet}} \times \frac{\text{span}}{\text{in feet}} \left(1 \times \frac{\text{span}}{10}\right).$$

The figures thus obtained give the approximate weight of the truss itself. To that must be added the weight of purlins, which may be directly calculated, also common rafters if any, and boarding and roof covering which may be taken from Tables I. and II.

Table III. is not intended for use in steel roof calculations for large spans.

Method of Calculation.

The method of calculation of the members of steel trusses is best shown by carefully studying the example which follows. The system adopted is first to work out the total dead load due to weight of roof covering and its supporting structure with the weight of the truss itself; then divide this total dead load by judgment into equivalent parts taken as acting on the panel points of the truss and draw a stress diagram for dead load.

The stress in each member due to dead load being thus obtained, that due to whatever dead load is assumed to be equal to the load due to workmen on the roof may be written down in direct proportion. The stresses due to wind are taken from the stress diagrams for wind, and the total direct stress in each member is then taken as that due to permanent load + workmen, or permanent load + wind, whichever may be the greatest; it being agreed that the maximum wind load will not occur simultaneously with that of workmen.

The stresses thus obtained form the basis of calculation for all the members and joints of the truss; but transverse stresses such as occur in a principal rafter when the purlins are not placed directly over the panel points of the truss, or in a tie bar carrying a ceiling, must also be taken into account, and may be found to very appreciably affect the design.

EXAMPLE IX.—A roof over a barrack block in a tropical country is to be constructed entirely of steel. Clear span 24′. Trusses at 7′ 6″ centre to centre. Angle steel purlins at 1′ centre to centre to carry double Allahabad tiling.

Pitch of roof 1 in 4 = slope 1 in 2.

Camber in tie rod $\frac{1}{40}$ span = slope in tie rod $\frac{1}{20}$.

Loads.

The permanent load on the roof consisting of double Allahabad tiling, steel purlins, and trusses may for purposes of calculation be taken at 40 lbs. per square foot subject to a check verification of the actual

weight of the steelwork after calculation. The occasional load due to workmen may be taken at 20 lbs. per square foot. The occasional load due to a horizontal wind pressure of 40 lbs. will give a load normal to the roof of 24 lbs. per square foot.

The length of the rafter on the slope (taking 25′ as the span for calculation) $= 12\frac{1}{2} \times 1{\cdot}11803 = 14'$ nearly. The total permanent load on one truss $= 2 \times 14 \times 7\frac{1}{2} \times 40 = 8{,}400$ lbs. vertical load.

The load due to workmen $= 4{,}200$ lbs. vertical load.

The load due to wind on one side of the roof and normal to the roof slope $= 14 \times 7\frac{1}{2} \times 24 = 2{,}520$ lbs.

The purlins are distributed along the length of the principal rafter at 1′ centre to centre, so that the load division may be taken as in *Figs.* 16A, 16B, 16C, and 16D, *Plate* IV.

Purlins. The purlins may be calculated as supported on each truss and uniformly loaded, though in practice long pieces of angle steel would probably be used continuous over two or more trusses, thus giving greater stiffness.

The load on each purlin due to permanent load and workmen is $7\frac{1}{2} \times 1 \times 60 = 450$ lbs.

Using the working stress for steel given in Table IX. we have :—

For strength

$$\frac{I}{y} = \frac{WL}{5} = \frac{450 \times 7\frac{1}{2}}{2240 \times 5} = {\cdot}3013.$$

For deflection

$$I = \frac{WL^2}{15} = \frac{450 \times 7\frac{1}{2} \times 7\frac{1}{2}}{2040 \times 15} = {\cdot}7534.$$

Referring to the tables of the British Standard Sections (which are found in convenient form in Dorman, Long & Co.'s *Pocket Companion*).

We find that a convenient and suitable section of angle steel is one $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$ weighing 5·89 lbs. per foot and having a section modulus of ·55 and the moment of inertia of ·962 : this section will therefore be adopted. The weight of the purlins is then 5·89 lbs. per square foot of roof surface.

Trusses. We may next draw the stress diagrams for the truss to obtain the direct stresses, *vide* the Figures on *Plate* IV. :—

(*a*). *Fig.* 16A for permanent load.
(*b*). *Fig.* 16B for wind on left, both ends fixed.
(*c*). *Fig.* 16C for wind on left, one end free.
(*d*). *Fig.* 16D for wind on right, same end free.

The stresses thus obtained are measured off and tabulated.

The column for stress due to workmen is in direct proportion to that due to permanent load ; and the maximum stress in any member is that due to permanent load + workmen, or permanent load + wind,

whichever is greatest in any particular case; the figures used being underlined.

The column for wind on the right with both ends fixed, or held by friction, is the converse of the preceding column and is merely entered for convenience of addition, a separate diagram not being needed :—

Table of Stresses.

TABULATION OF STRESSES.

	Member of Roof.	Stress due to Permanent Load.	Stress due to Workmen.	Stresses due to Wind.				Maximum Stresses.	Stresses for Calculation.
				Reactions Parallel.		Reaction Free and Vertical.			
				On Left.	On Right.	On Left.	On Right.	lbs.	tons.
Principal.	CH	7850	3925	2500	1700	2550	1625	11775	5·26
	DK	6900	3450	2500	1700	2550	1625	10350	4·62
	EM	6900	3450	1700	2500	1725	2425		
	FN	7850	3925	1700	2500	1725	2425		
Tie bars.	GH	7000	3500	2750	1200	3150	375	10500	4·69
	GL	4500	2250	1150	1150	1475	350	6750	3·01
	GN	7000	3500	1200	2750	1550	1900		
	KL	2650	1325	1625	50	1650	25	4300	1·92
	ML	2650	1325	50	1625	100	1550		
Struts.	HK	1900	950	1260	0	1260	0	3160	1·41
	MN	1900	950	0	1260	0	1260		

The principal rafter will be of the same section throughout, so only the portion CH in which the greatest stress occurs need be calculated.

The strut MN is complimentary to HK.
The tie ML „ „ KL.
The tie GN „ „ GH.

Principal Rafter.

The greatest direct stress in the principal rafter found from the stress diagrams is 5·26 tons in CH. Owing to the spacing of purlins along the length of the rafter this member is also under transverse load.

Considering the portion CH the maximum transverse load is that due to the normal compotent of the permanent load which may be taken at $40 \times \text{cosine } 26^\circ\ 35'' = 40 \times \cdot 89441 = 36$ lbs. per square foot nearly, and to this must be added the normal wind pressure which amounts to 24 lbs. per square foot, so that the total transverse load is 60 lbs. per square foot. The transverse load on CH is therefore a distributed load $= 7 \times 7\frac{1}{2} \times 60 = 3{,}150$ lbs. $= 1{\cdot}406$ tons.

And we may consider that as regards tranverse load the principal

rafter is a beam fixed at each end and continuous over the strut, so that the maximum bending moment

$$M_{ff}=\frac{W/}{12}=\frac{1\cdot406\times7\times12}{12}=9\cdot842 \text{ inch tons.}$$

And the moment of resistance of the member to cross breaking is

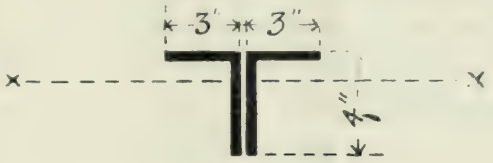
$$Mr=\frac{rI}{y}$$

$$\therefore \qquad r=\frac{9\cdot842\times y}{I}.$$

As the purlins give good lateral support, being fastened to angle cleats on the principal by bolts or rivets, they will prevent the principal from buckling sideways ; a deep section of principal with a good amount of metal in the stem is evidently required. For this purpose a principal built up of two unequal angles appears best suited.

We now select from the list of British Standard Sections what appears to be a suitable size and test it for combined stress.

Let us try two angles each $4''\times3''\times\frac{3}{8}''$.



The following data are then obtained from Dorman, Long & Co.'s *Pocket Companion.*

Area of section $A=2\times2\cdot485=4.97$ square inch.

Weight per ft. run $=2\times8\cdot45=16\cdot90$ lbs.

Moment of inertia $I=2\times3\cdot89=7\cdot78$

Section modulus $\frac{I}{y}=2\times1\cdot42=2\cdot84$

Both taken with respect to the horizontal axis X—X.

Radius of gyration about horizontal axis—

$$k=\sqrt{\frac{I}{A}}=\sqrt{\frac{7\cdot78}{4\cdot97}}=\sqrt{1\cdot56}=1\cdot25.$$

Greatest unsupported length in inches $l=84$

Then $$\frac{l}{k}=\frac{84}{1\cdot25}=67\cdot2.$$

The safe stress in the member taken as with ends fixed is from Table X.$=4\cdot71$ tons per square inch.

The maximum stress applied to the member is :—

r_c due to direct compression $=\frac{5\cdot26}{4\cdot97}=1\cdot06$

r due to transverse stress $=\frac{9\cdot842}{2\cdot84}=3\cdot46$

Total $4\cdot52$ tons per square inch.

The combined stress is therefore just below that shown to be admissible by Table X., so that the section chosen is suitable.

The principal rafter may therefore be made of two angle steels each $4'' \times 3'' \times \frac{3}{8}''$, which will make a convenient section for structural purposes.

NOTE.—It may here be noticed what a large increase in the size of the principal rafter is caused by the transverse stress due to the distribution of the load along its length. If the purlins had been arranged so as to rest only at the panel points, the principal rafter would be in direct compression only, and a much smaller section would have sufficed. In this particular instance however the design is arranged intentionally so as to avoid the necessity for timber common rafters, and the arrangement adopted is probably on the whole the most economical.

Struts. The direct stress in the struts found from the stress diagram is 1·41 tons in H.K.

The struts are in direct compressive stress only.

The length of the strut is $3' = 36''$.

Assume a section of angle steel $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$.

Then area of section = 1·733 square inch.

Weight per ft. run = 5·89 lbs.

Least radius of gyration $k = \cdot 48$.

$$\frac{l}{k} = \frac{36}{\cdot 48} = 75 .$$

The safe stress in the member taken with ends rounded is from Table X. = 3·37 tons per square inch.

The maximum stress applied to the member is only

$$\frac{1 \cdot 41}{1 \cdot 733} = \cdot 8 \text{ tons per square inch.}$$

The section chosen is therefore amply strong; but from the practical points with regard to riveting, given in Part II., a less width than $2\frac{1}{2}''$ cannot conveniently be used with $\frac{3}{4}''$ rivets, and to allow for corrosion a less thickness than $\frac{3}{8}''$ is undesirable in steel roof work generally. Further the stress is applied eccentrically so that a further margin of safety over and above that given by considering the ends rounded is usually desirable in the struts of a steel roof.

The struts will therefore be made of angle steel $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$.

Tie Bars. The tension in tie bar GH is 4·69 tons.

The tension in tie bar GL is 3·01 tons.

The tension in tie bar KL is 1·92 tons.

The tie bars are in direct tensile stress only.

The safe stress in tension is 8 tons per square inch (Table IX.)

A tie bar $2\frac{1}{2}'' \times \frac{3}{8}''$, after deducting $\frac{3}{4}''$ for one rivet hole, will give an effective section of $1\frac{3}{4} \times \frac{3}{8} = \frac{21}{32}$ square inch, and will bear a safe stress of $\frac{21}{32} \times 8 = 5 \cdot 25$ tons.

A tie bar $2\frac{1}{4}'' \times \frac{3}{8}''$, after deducting $\frac{3}{4}''$ for one rivet hole, will give an effective section of $1\frac{1}{2} \times \frac{3}{8} = \frac{9}{16}$ square inch, and will bear a safe stress of $\frac{9}{16} \times 8 = 4{\cdot}5$ tons.

For the practical reasons noted above this is the least size that may conveniently be used.

As it is also undesirable to complicate the construction by using more different sections than is really necessary it will be as well to make all the tie bars throughout of the one section, viz.: $2\frac{1}{2}'' \times \frac{3}{8}''$ which is required for GH.

All the tie bars will be of $2\frac{1}{2}'' \times \frac{3}{8}''$ flat steel.

Joints. Referring to Table XI. we see that for the joint between the principal rafter CH and tie bar GH three $\frac{3}{4}''$ rivets will be suitable; these rivets being in double shear as regards the principal and in single shear as regards the tie bar if a single cover plate is used.

For all other joints two $\frac{3}{4}''$ rivets will be sufficient.

Cover Plates. The cover plates may be single so as to fix between the two angles forming the principal in the case of the joints with that member. Table XI. shows that $\frac{3}{8}''$ metal will give sufficient bearing value.

General Design. The truss may now be designed in detail as shown in *Fig.* 16; the joints may be riveted or bolted as may be most convenient according to local practice. Riveting is best if pneumatic or hydraulic riveters can be employed, as it gives tighter joints, and the plates being drawn closely together the friction between them gives an additional factor of safety. If the riveting can only be done by hand and by comparatively unskilled workmen the holes are not properly filled and the strength of the metal is apt to be reduced so that bolting is more reliable. The purlins would in any case probably be best bolted to the angle cleats on the principal rafter so as to allow for a little freedom of movement.

Material Required. The estimated amount of material and the weight of one truss will be as follows:—

Angle steel $4'' \times 3'' \times \frac{3}{8}''$.

Principal rafters	$4 \times 14'\ 0'' = 56$	$59'$ at 8·45 lbs. = 499
Angle shoes	$4 \times 0'\ 9'' = 3$	

Angle steel $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$.

Struts	$2 \times 3'\ 3'' = 6\frac{1}{2}$	$20\frac{1}{2}'$ at 5·89 lbs. = 120
Angle cleats	$28 \times 0'\ 6'' = 14$	

Flat steel $2\frac{1}{2}'' \times \frac{3}{8}''$.

Ties GH and GN	$2 \times 7'\ 6'' = 15$	$39\frac{1}{4}'$ at 3·19 lbs. = 125
Ties KL and ML	$2 \times 7'\ 3'' = 14\frac{1}{2}$	
Tie GL	$1 \times 9'\ 9'' = 9\frac{3}{4}$	

Cover plates of $\frac{3}{8}''$ steel allow 5 square feet at 15·30 lbs. = 76

Add for rivets or bolts, 43, No., at average ·4 lbs. each = 18

Total weight of one truss 838 lbs.

Area covered by one truss = $24' \times 7'\ 6'' = 180$ square feet.

Weight of truss per square foot of area covered = 4·66 lbs.

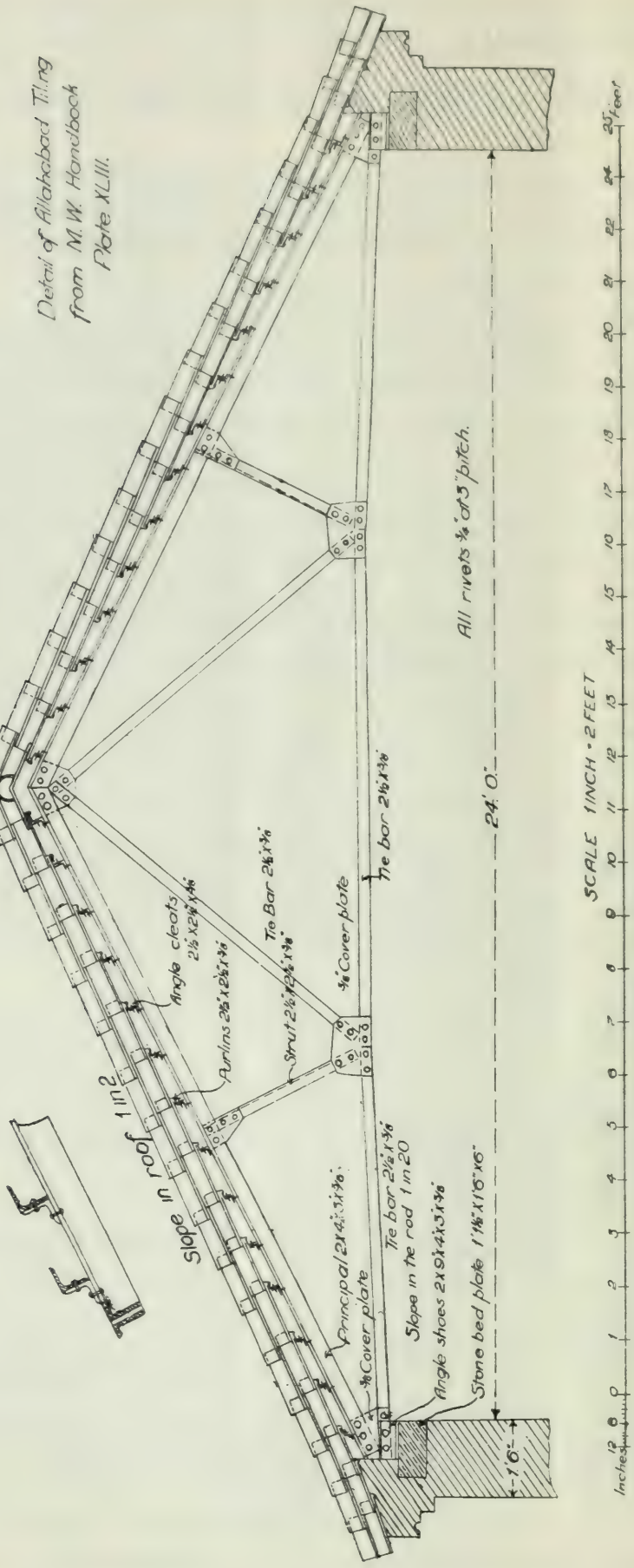


FIG. 16.—*A Steel Roof Truss 24' Clear Span.—Trusses 7' 6" c. to c. to carry Double Allahabad Tiles.*

Method of Sections.—With trusses of the type shown in *Fig.* 17 and similar types the stress diagram cannot be drawn in the ordinary way. On arriving at the point CDGHIJ, there are two unknown stresses at this joint and at the lower joint LIHM, and the diagram cannot be continued.

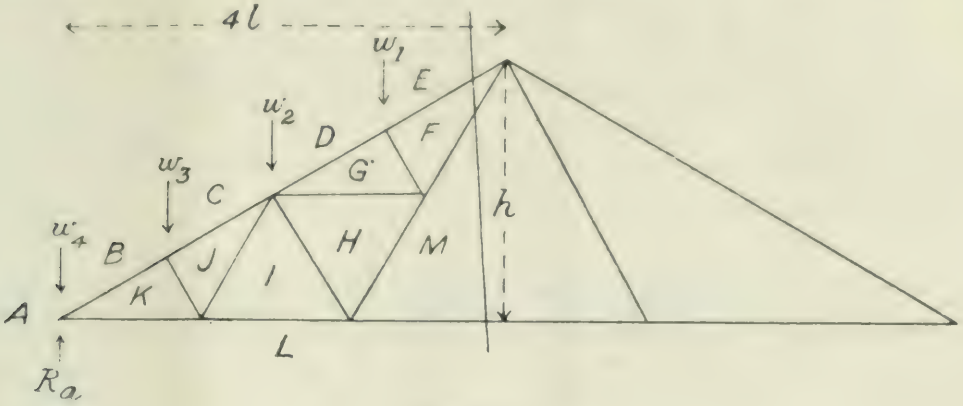


FIG. 17.

If we can find the stress in LM, we can go on and find HI graphically, and so complete the joint CDG, etc.

Imagine a section passed through the truss cutting the spaces EFM. The left-hand side of the truss is kept in equilibrium by the external forces and the forces acting in these three bars EF, FM, LM.

Take moments about the apex, measuring the lever arms off the diagram and so get an equation in which LM is the only unknown.

Thus

$$\mathrm{LM} \times h + \mathrm{R}a \times 4l = w_1 \times 4l + w_3 \times 3l + w_2 \times 2l + w_1 \times l.$$

NOTE.—Take the section so as to cut three bars only, it need not be a straight line. Take moments about the intersection of any two of these bars, to find the stress in the third.

This method is a useful check on results found graphically, and should always be applied as a test in working out any truss.

Polar Diagrams and Funicular Polygons.—Useful for finding the reactions at abutments in cases of:—

(1). Unsymmetrical loading.
(2). Saw tooth roofs.
(3). Hanging loads, such as machinery shafting.

Method.—(1). Produce the lines of the external forces downwards.
(2). Draw a load line $abc\ldots$ ($ab = w_1$ $bc = w_2$, etc.).
(3). Take any point o, and join ob, $oc\ldots$of. Do not join oa or og as these are not required.
(4). Take a point in W produced and draw b^1c^1 parallel to ob, c^1d^1 parallel to oc, etc.
(5). Join b^1g^1, and draw oh parallel to it in the polar diagram.

Then $ah = \mathrm{R}a$, $gh = \mathrm{R}b$, and the stress diagram can be drawn on the left of the load line already drawn.

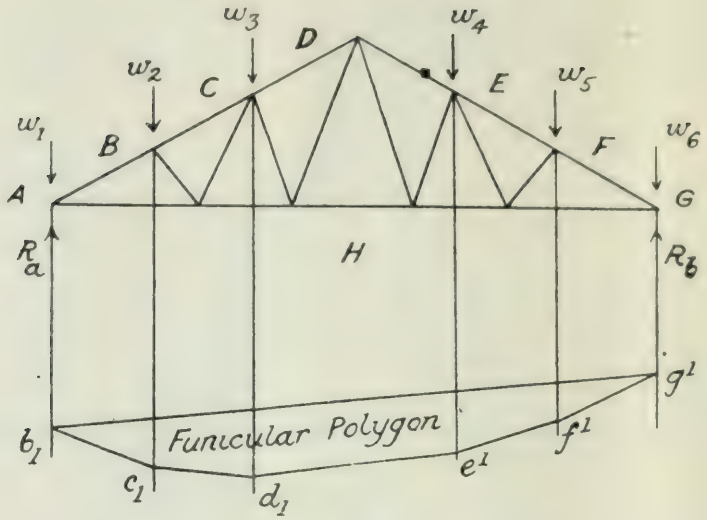


FIG. 18.

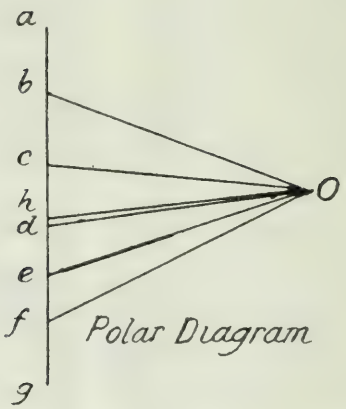


FIG. 19.

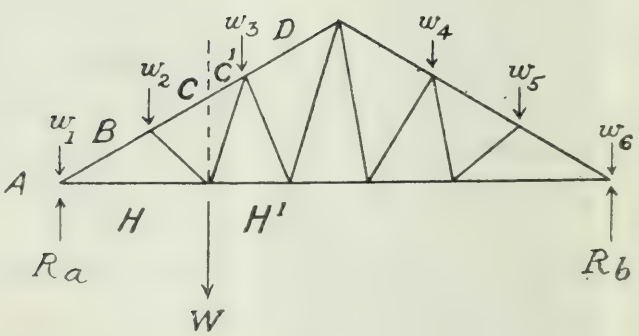


FIG. 20.

It is sometimes simpler to find R*a* and R*b* by an equation of moments.

If a *Hanging load* is suspended from the lower joint of a truss this method may be used.

For the polar diagram imagine the hanging load transferred to the principal rafter and applied as a superimposed load at X, thus dividing the space C into C and C^1.

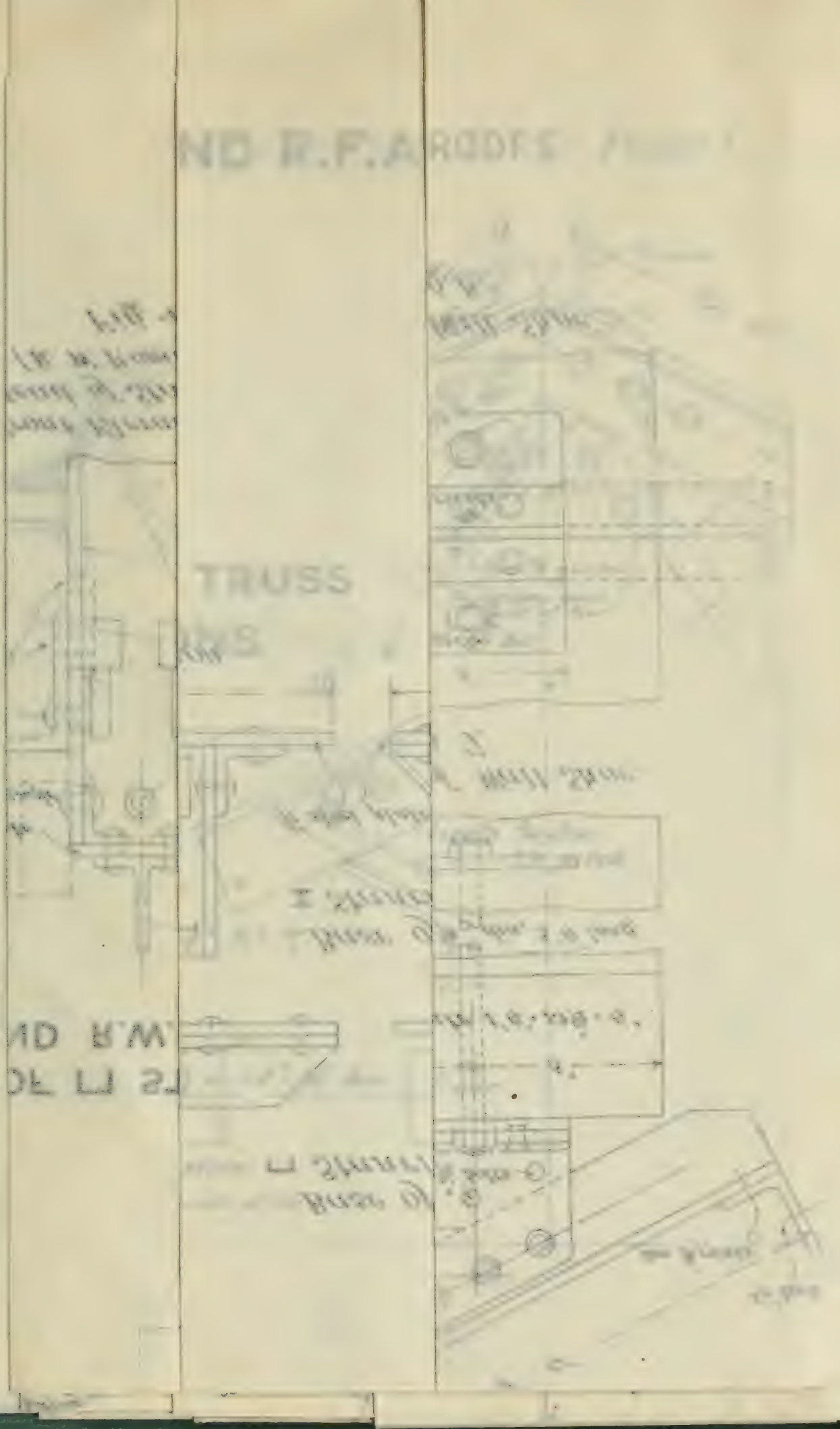
Then draw the polar diagram as before. But note that the load line thus drawn will not do for the stress diagram. Another load line must be drawn, considering the load hanging from the lower joint; the space C_1 will coalesce with C, but H will be divided into H and H^1.

Practical Details.

After the size of members and the correct proportions of the joints have been settled by calculation, the design of the practical details of construction has to be considered.

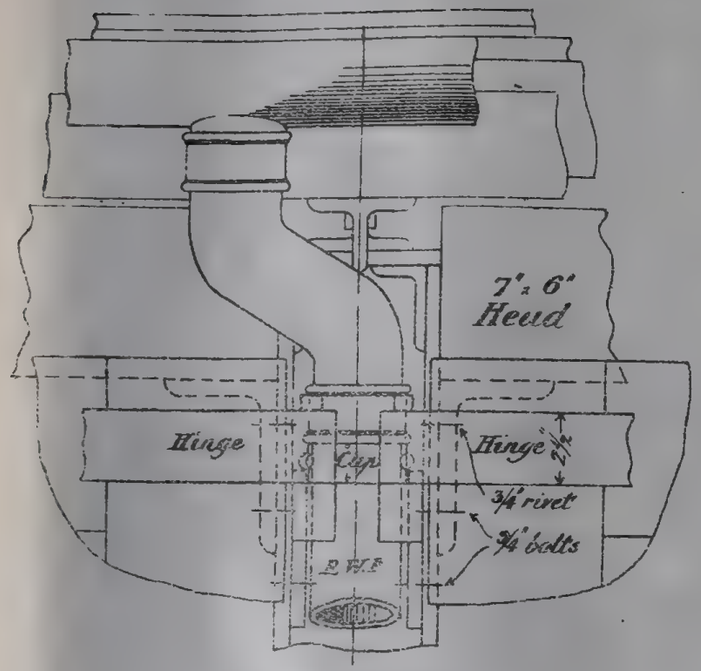
Existing roofs of modern design are the best guide, and for this purpose three designs are shown, viz.:—

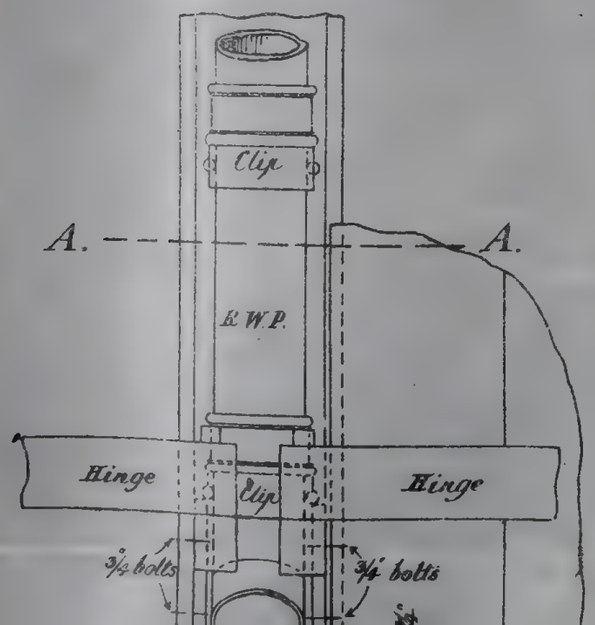
Plate I.—A gunshed roof of 40′ span.
Plate II.—A riding school roof of 73′ span.
Plate III.—A saw tooth roof of 15′ spans.

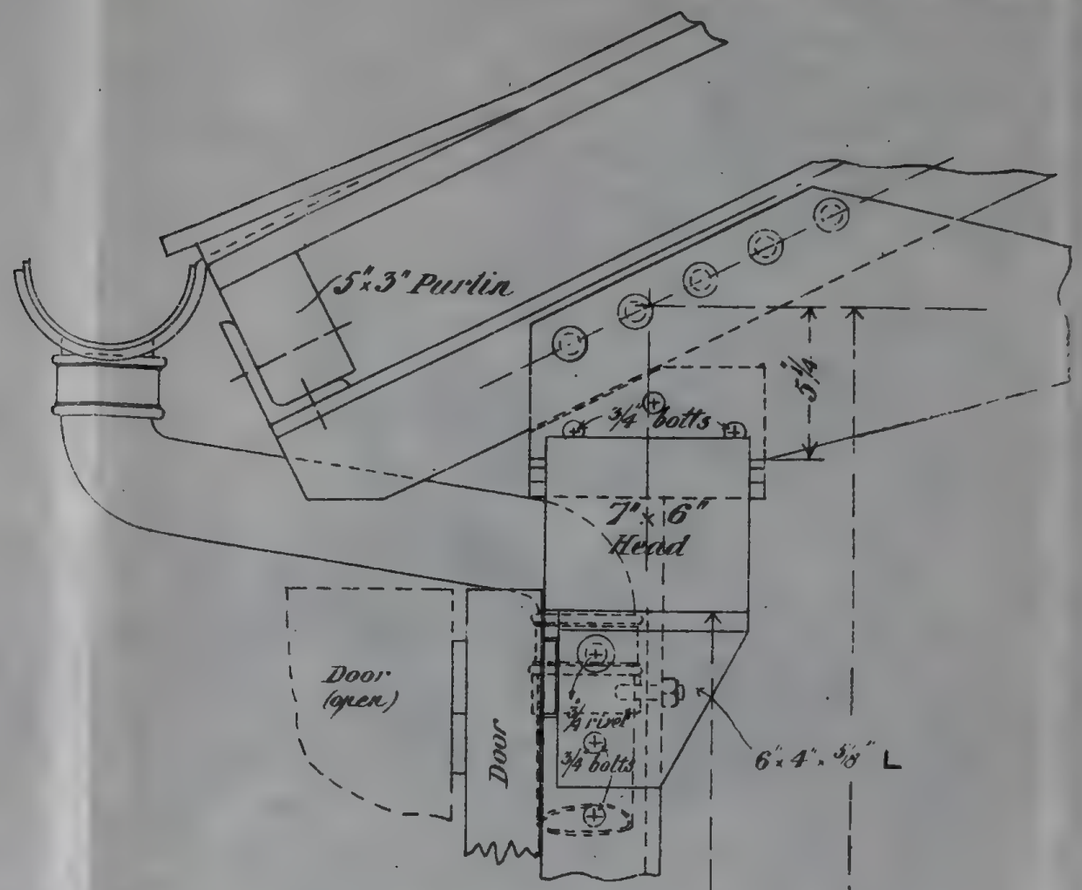


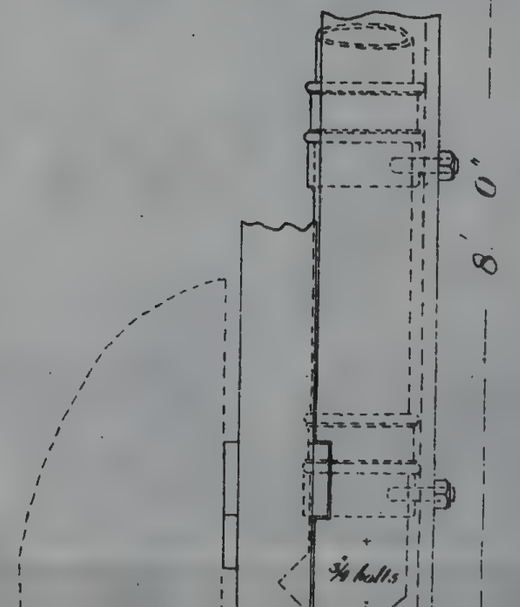
GUN AND WAGON SHED FOR R.H.A.

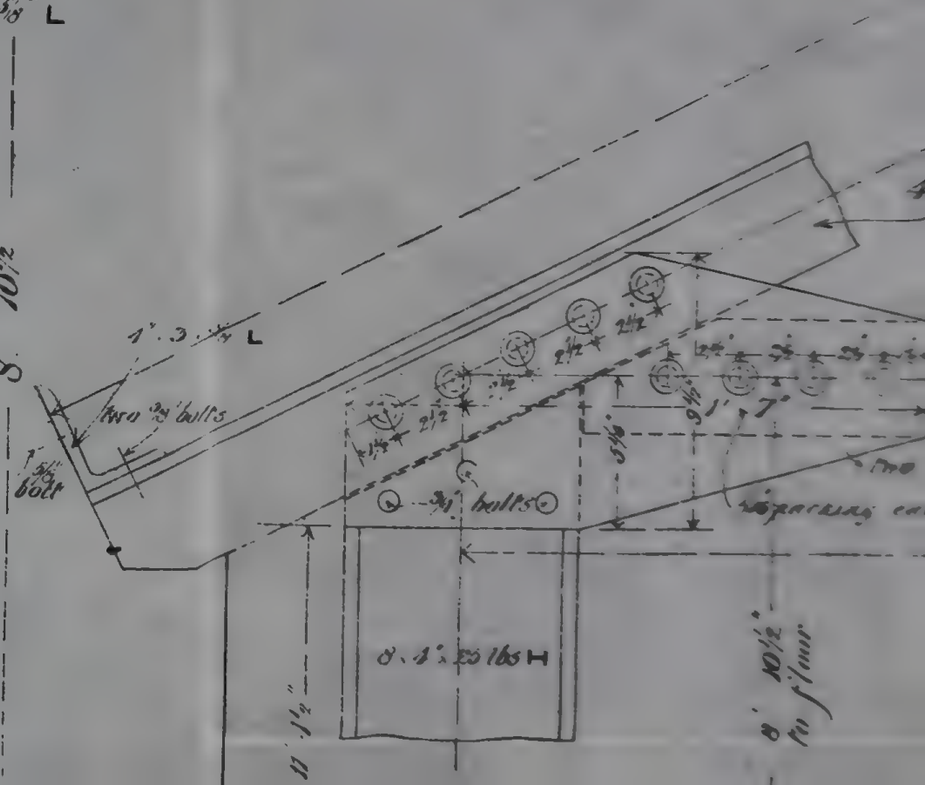
DETAIL OF STEE
AND STANCH

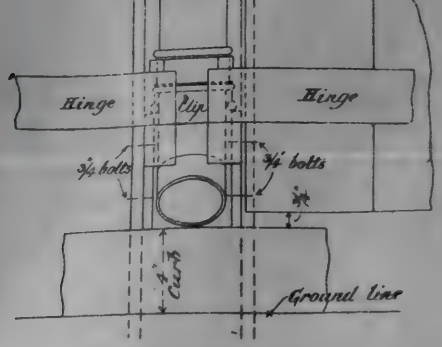






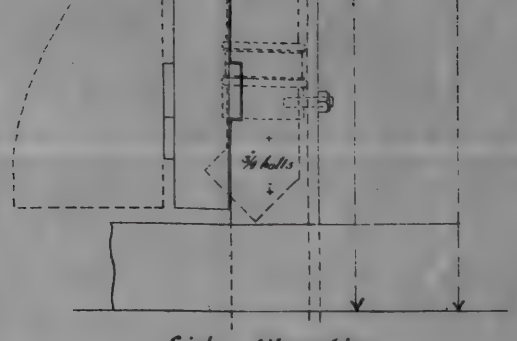






Front Elevation shewing R.W.P.

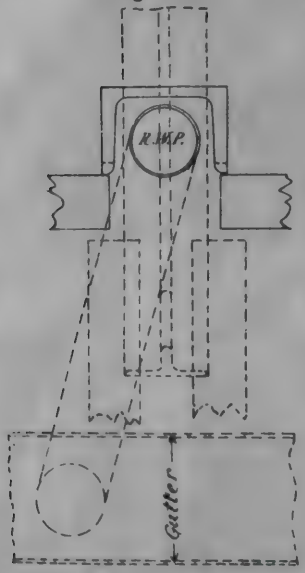
Fig. 1.



Side Elevation shewing R.W.P.

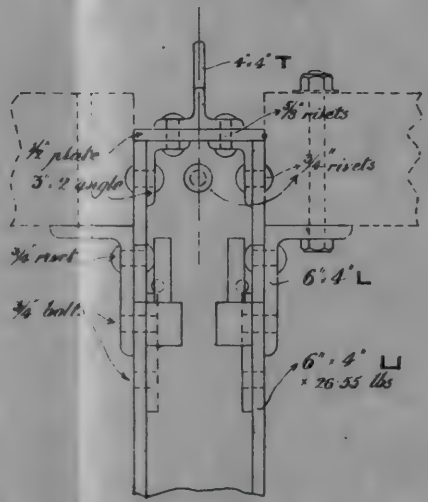
Fig. 2.

DETAILS OF ⊓ STANCHION AND R.W.P.



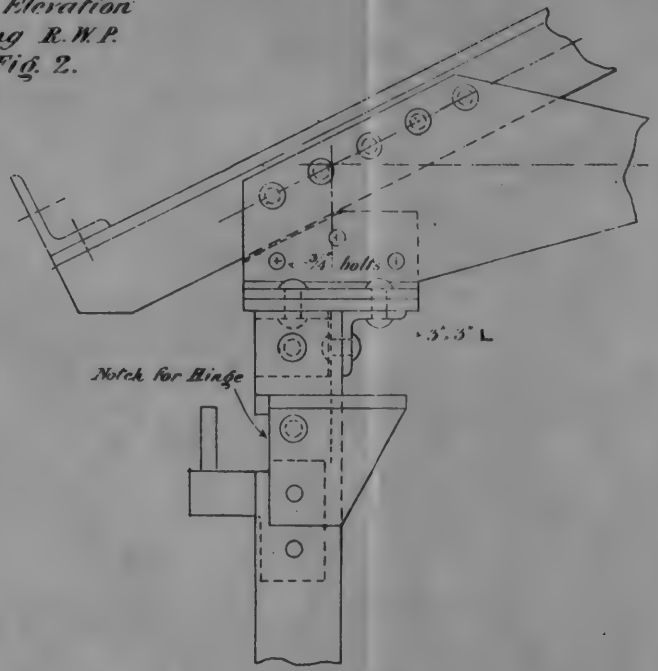
Plan on line A.A.

Fig. 3.



Front Elevation of Head of Stanchion (R.W.P. omitted)

Fig. 4.

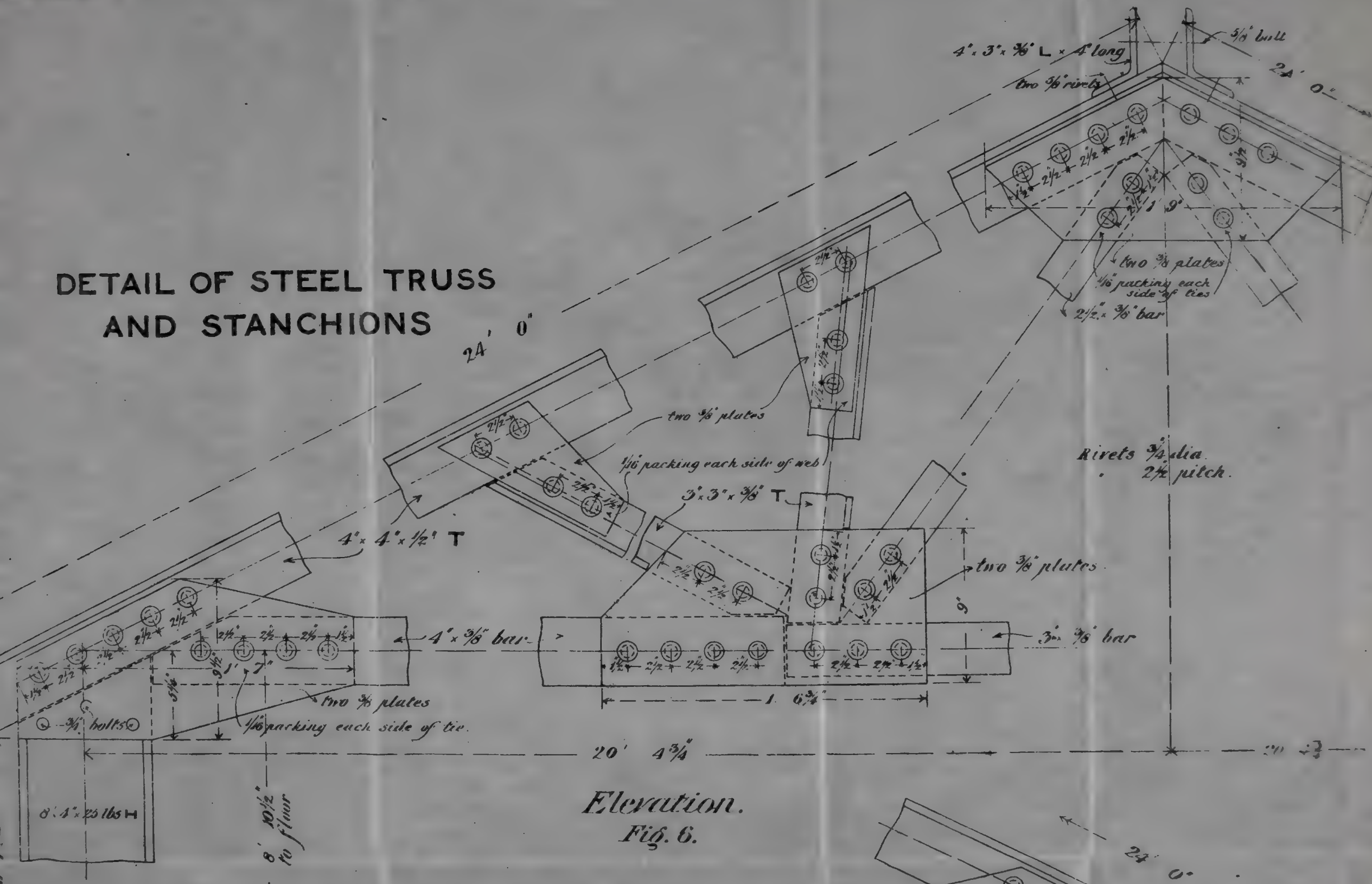


Side Elevation of Head of Stanchion (R.W.P. omitted)

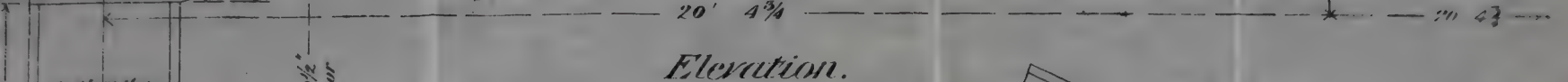
Fig. 5.

SHED FOR R.H.A. AND R.F.A.

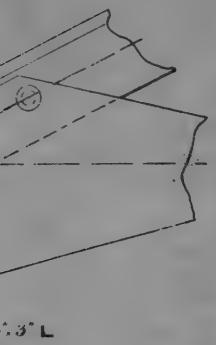
DETAIL OF STEEL TRUSS AND STANCHIONS



Elevation.
Fig. 6.



Elevation.
Fig. 6.



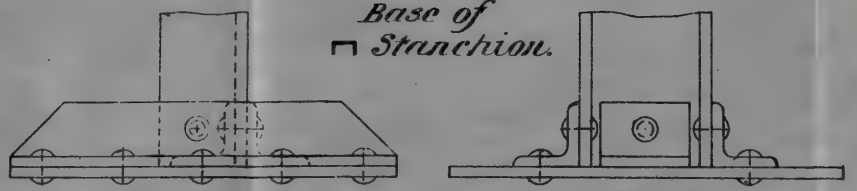
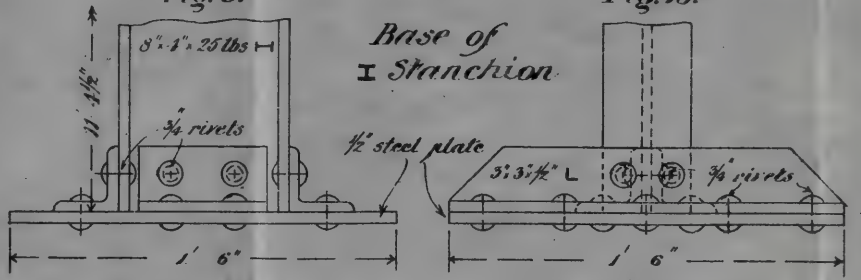


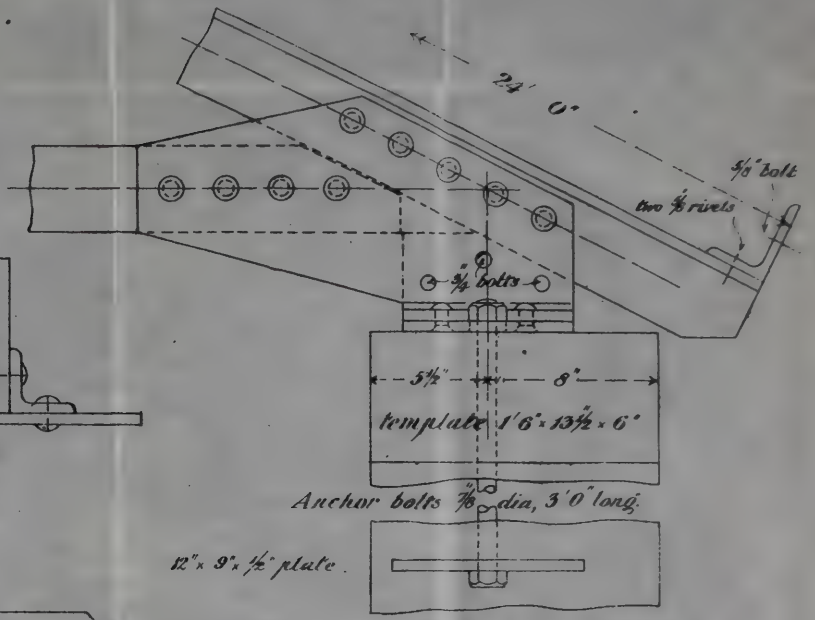
Fig. 9.

Fig. 10.

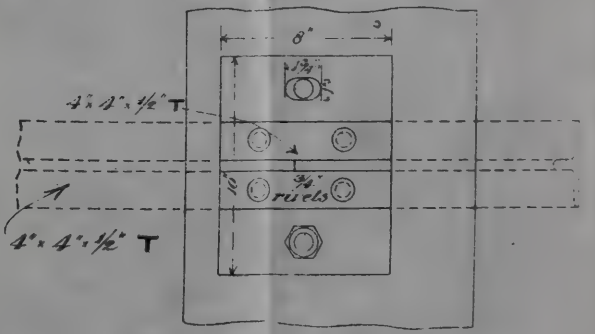


Side Elevation.
Fig. 11.

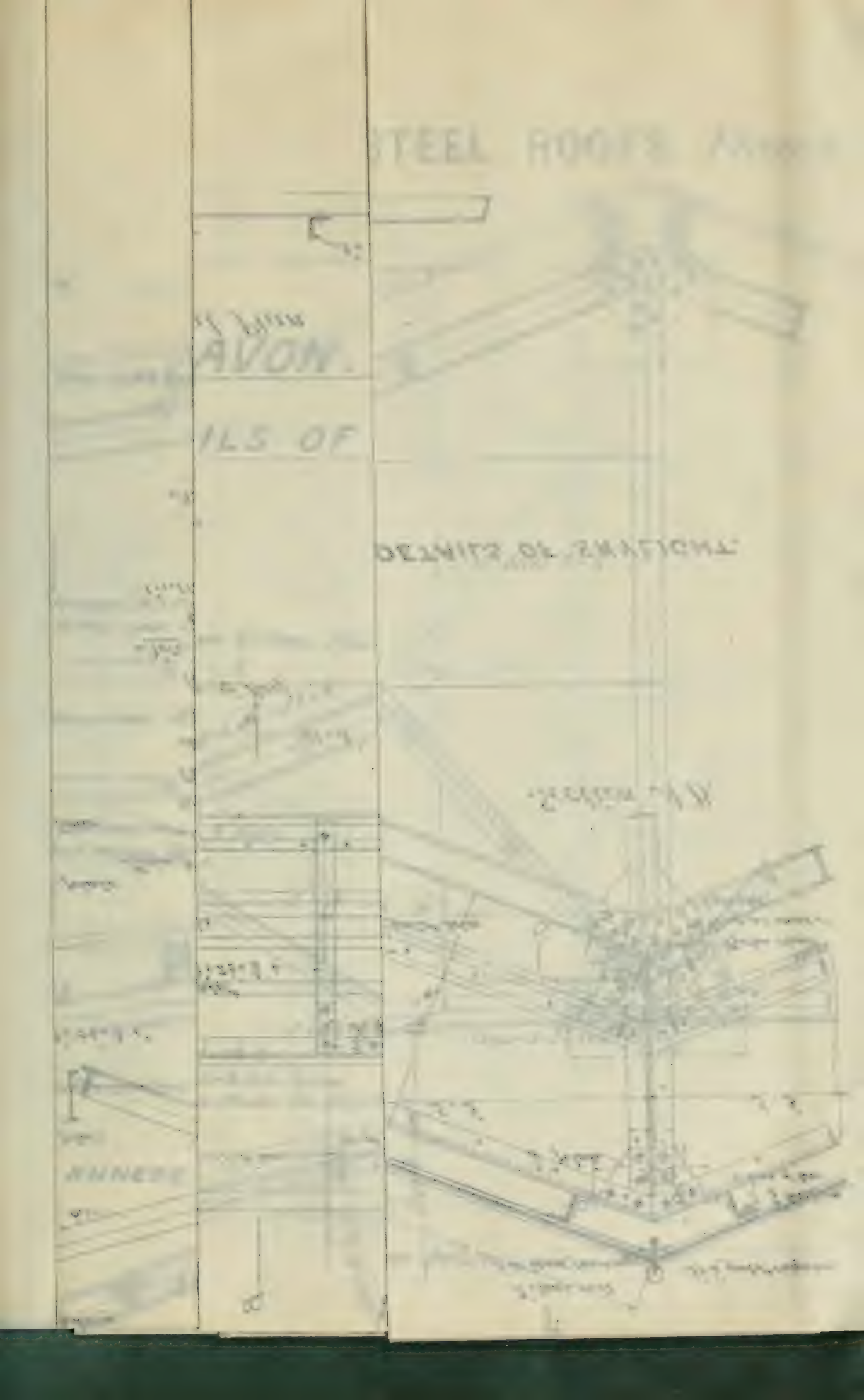
Front Elevation.
Fig. 12.



Elevation of Wall Shoe.
Fig. 7.



Plan of Wall Shoe.
Fig. 8.



NET

12'.6"

4'.6"

2'.0"

Purlins 7×3

Purlin cleats 4×3×⅜

⅞ bolt 1¼ long

4'.6"

B.S.C. 5×2½×11 lbs

⅜ rivet

2'.0"

10½

⅜ plate

3×2½×⅜ L

Lattice Stringer between Stanchions
See Dwg. 39436.

⅜ plate

3×2½×⅜ L

BSB 15 9×4×21 lbs

⅞ bolts

⅜ plate

Channel Stringers B.S.C. 9. 7×3×17¼ lbs
4 to this, each 22'.5¾ long.

⅞ bolts

1'.3×7'.⅜ plate

3 Sets of Stanchions, Rafters, &c. to this

17'.5⅜

22'.10½

Stanchions

9½×7×⅜ plate

1 Set to this for end bay.
1 do. as but reverse handed as above.

5×3×⅜ L

Plate 1'.3×1'.3×⅜

Purlin cleats to Ridge with
these on Rafters above

12'.3⅜

Slope as above

5×2½×11 lbs

4 Intermediate Rafters to this.

⅞ bolts

7×3 Stringer

9" wall

DETAILS OF ANNEXE.

Purlins 7×3

5'.3"

7×3 Channel

Taper liner

2'.0"

Purlin cleats 4×3×⅜
5¾ long

2-⅞ bolts 4½ long

3×3×⅜ do.

1'.0"

2 3×3×⅜ L

Lattice Purlin

3×⅜ Bar between L's

1×⅜ bar

2 angles 4×3×½ B.S.U.A. 11

16 gauge galv.d steel flashing fixed to underside of glazing bar

14'.7"

⅞ rivets

5×3×½ distance pieces

Joints in Channels

2-3×3×⅜

2½×⅜ bar

⅜ plate

⅞ bolts

3×2½×⅜ L

Joints in

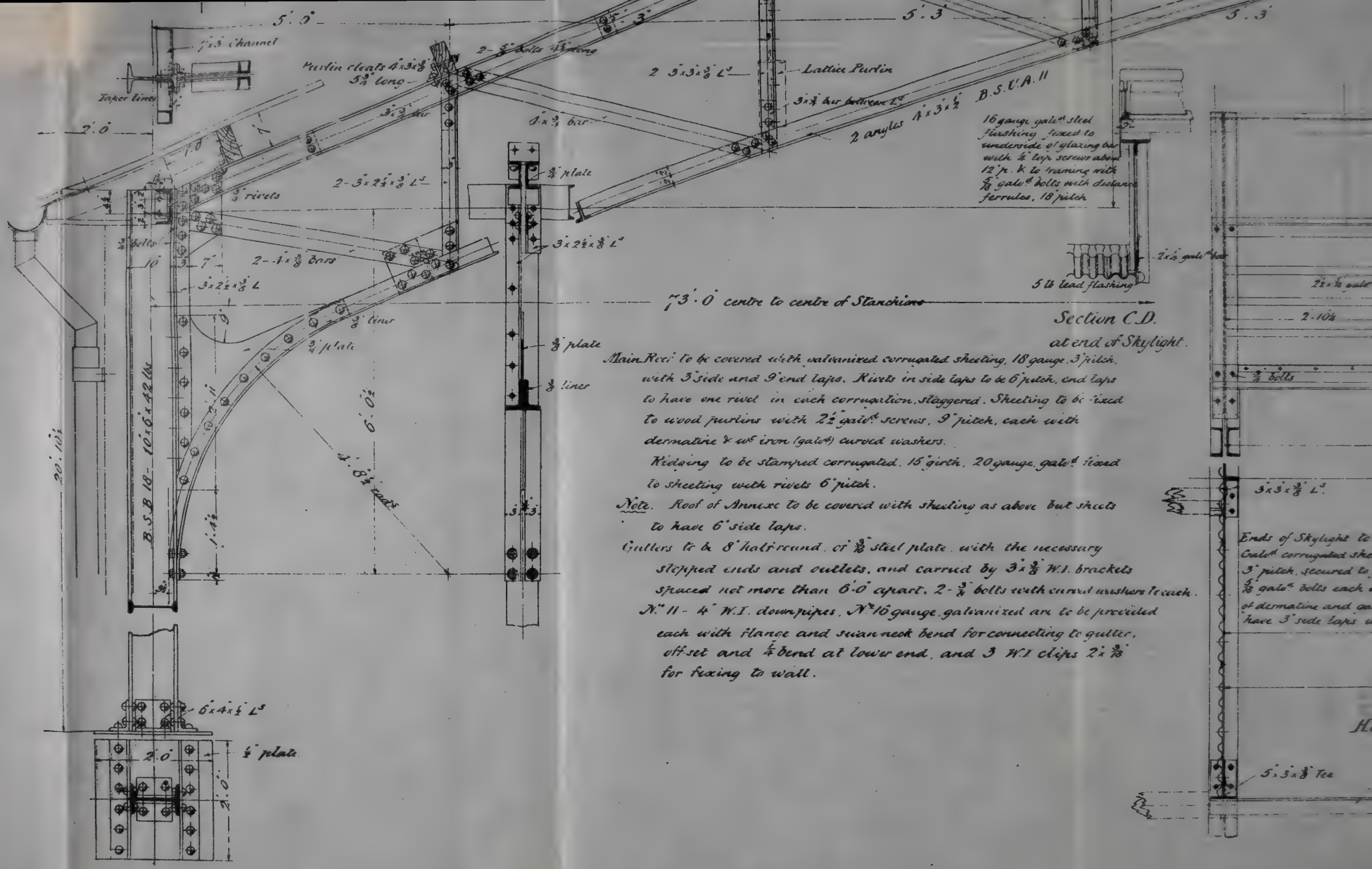
2½×⅜ bar

5'.3"

5'.3"

Note. Except when otherwise sta

Connection plates ⅜ thi



Main Roof to be covered with galvanized corrugated sheeting, 18 gauge, 3" pitch, with 3" side and 9" end laps. Rivets in side laps to be 6" pitch, end laps to have one rivet in each corrugation, staggered. Sheeting to be fixed to wood purlins with 2½" galvd screws, 9" pitch, each with dermatine & wr iron (galvd) curved washers.

Ridging to be stamped corrugated, 15" girth, 20 gauge, galvd fixed to sheeting with rivets 6" pitch.

Note. Roof of Annexe to be covered with sheeting as above but sheets to have 6" side laps.

Gutters to be 8" half round, of ⅛" steel plate, with the necessary stopped ends and outlets, and carried by 3"x⅜" W.I. brackets spaced not more than 6'.0" apart. 2-⅝" bolts with curved washers to each.

Nos 11 - 4" W.I. downpipes, No 16 gauge galvanized are to be provided each with flange and swan neck bend for connecting to gutter, off-set and ¼ bend at lower end, and 3 W.I clips 2"x⅜" for fixing to wall.

NETHERAVON. RIDING SCHOOL.

DETAILS OF ROOF TRUSSES. &c

Except when otherwise stated all bolts are $\frac{3}{4}$" diam. Shewn thus-✦
rivets - $\frac{3}{4}$"
Connection plates $\frac{3}{4}$" thick.

Cover 2-5½ x 5¾ x ½

Joints in Channels

¾" plate

2-2½

5'.3"

5'.3"

5'.3"

5'.3"

5 - 3 x ½ distance pieces

3" x 2½ x ⅜ L

2 - 3" x 2½ x ⅜ L

2 - 3" x 3" x ⅜ L°

Plate 9½ x 6¾ x ⅜
Holes for ¾" bolts fixing
Lattice Purlin Sec. Dwg. 39436.

2½ x ⅜ bar

¾" plate

¾" rivets

¾" bolts

Cover 2-2½ x 6¾ x ½

Joints in angles

1-11½

110 ft² rad.

Lattice purlin
Sec. Dwg. 39436

Plate 9½ x 6¾ x ⅜

2 - 3" x 2½ x ⅜ L

2½ dia. x ¾ distance piece

¾" rivet

2 - 3" x 3" x ⅜ L°

2 bars 4" x ⅜

8'.3"

Joints in angles

Cover 2-2½ x 6¾ x ½

1-11½

50 ft² rad.

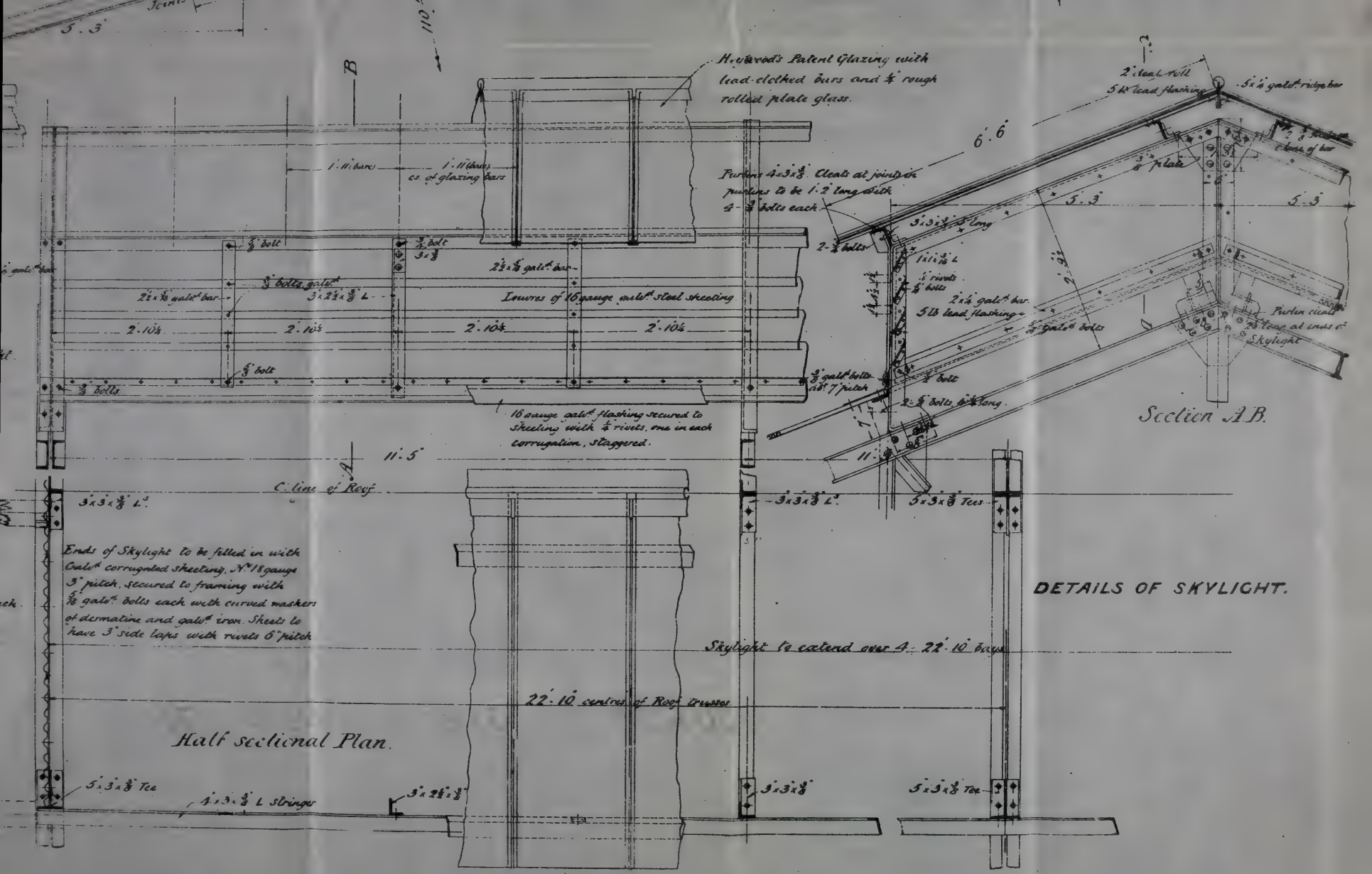
9 Roof Trusses with Stanchions to this.

5 trusses to be prepared to take Skylight as shewn below.

B

Heywood's Patent Glazing with
lead-clothed bars and ¼" rough

2" tee rail

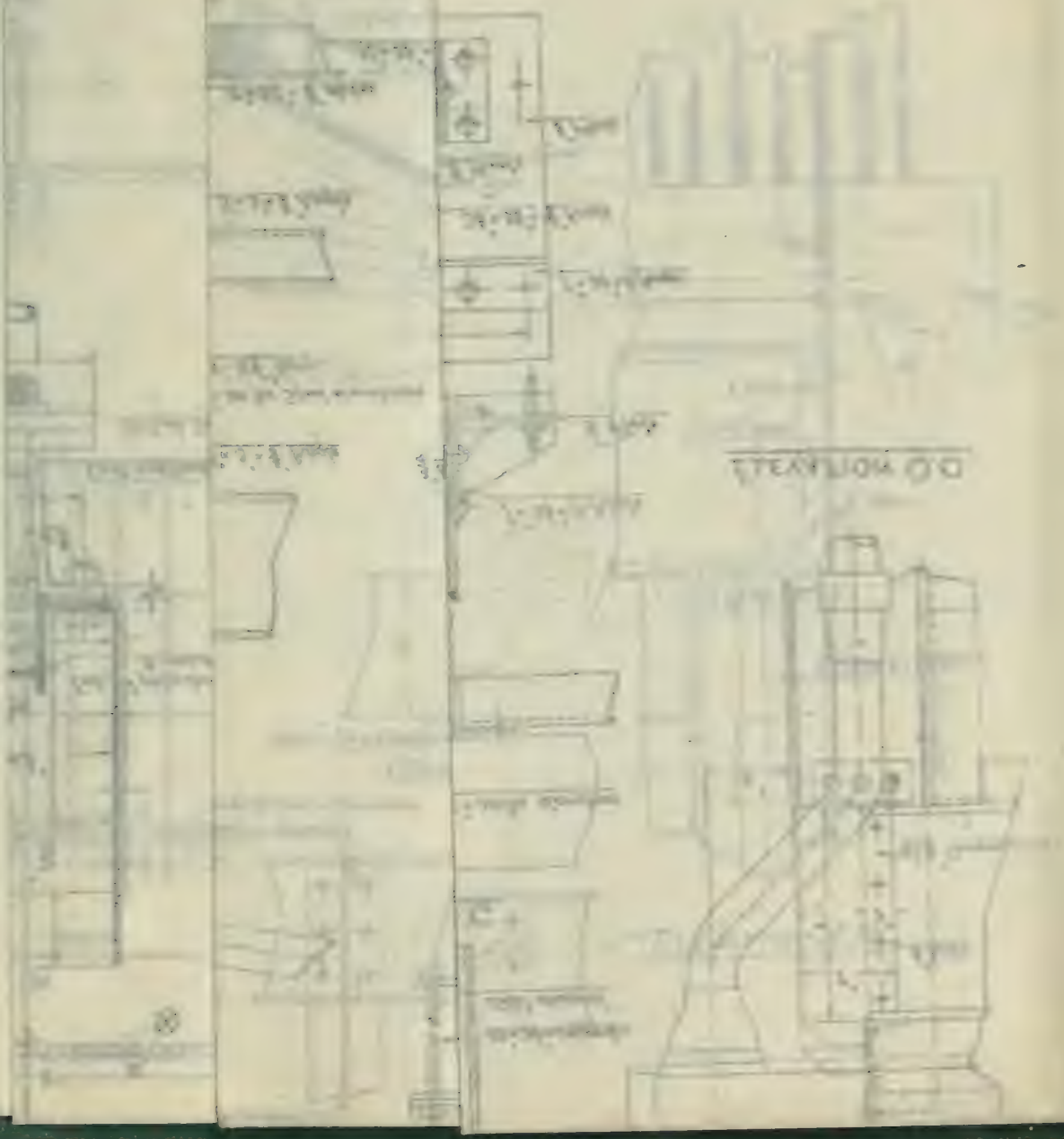

Hartwood's Patent Glazing with lead-clothed bars and ¼" rough rolled plate glass.
Louvres of 16 gauge galvd steel sheeting
16 gauge galvd flashing secured to sheeting with ¼" rivets, one in each corrugation, staggered.
Purlins 4"x3"x⅜" Cleats at joints of purlins to be 1'-2" long with 4-¾" bolts each
2" clear roll
5lb lead flashing
5"x4" galvd ridge bar
Section A.B.
Ends of Skylight to be filled in with Galvd corrugated sheeting, No 18 gauge 3" pitch, secured to framing with ⅜" galvd bolts each with curved washers of dermatine and galvd iron. Sheets to have 3" side laps with rivets 6" pitch
Skylight to extend over 4 - 22'-10" bays
22'-10" centres of Roof trusses
Half Sectional Plan.
DETAILS OF SKYLIGHT.

ONT

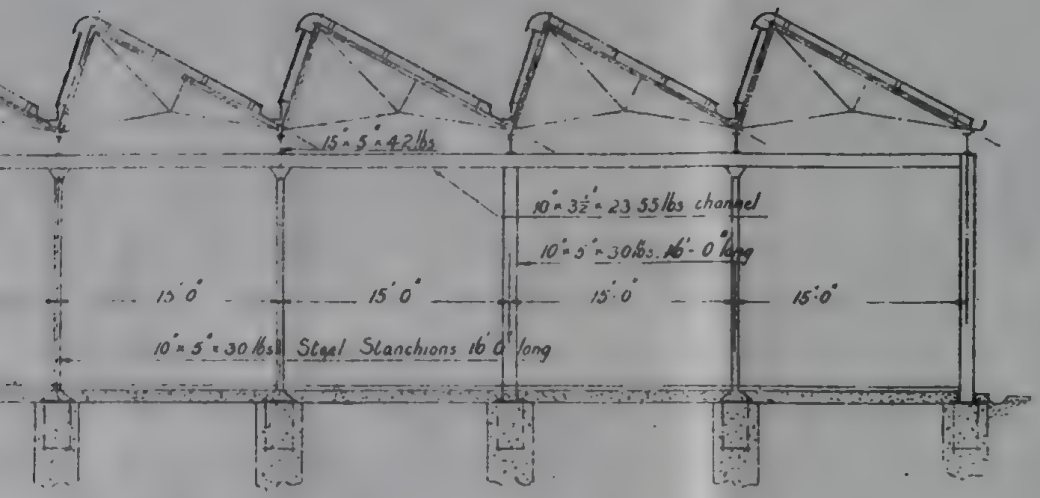
SHOPS

STEEL ROOFS

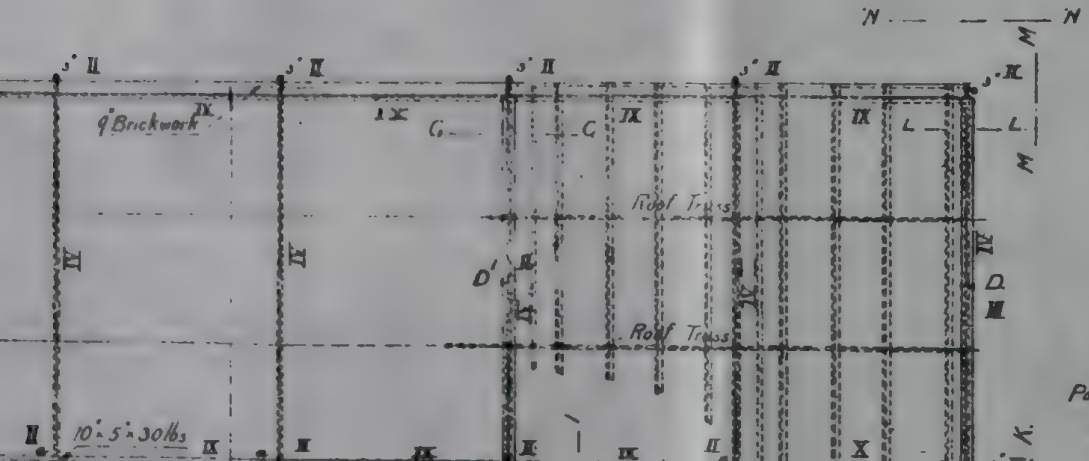
Plate III



R.E. CENTRAL WORKSHOPS



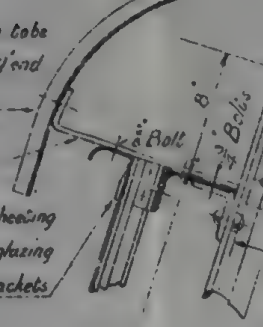
SECTION X.X.



Ridge 18 G. Corrugated steel sheeting to be curved to 13" radius with 6" side and 9" end lap. Riveting as for other sheets

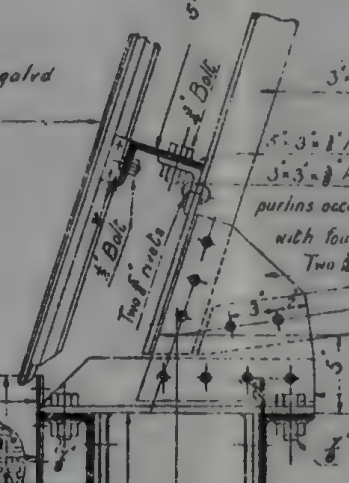
3" x ⅜" W.I. brackets 2'-1" apart to support ridge

Galvanized steel sheeting No 18 G. bedded over glazing bars and jointed at brackets



Heywoods glazing. ¼" rolled plate glass on galvd steel bars 2'-1" centres except at ends

All bolts and rivets ¾" dia unless otherwise specified



¼" Gutter with necessary joint covers stopped ends, and outlets for 3" pipes

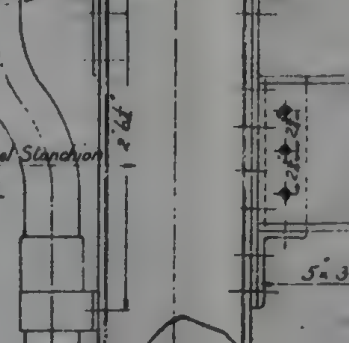
Cast iron rose to be fixed over all down pipes except at 'a'

13 - 2'-1" Brackets

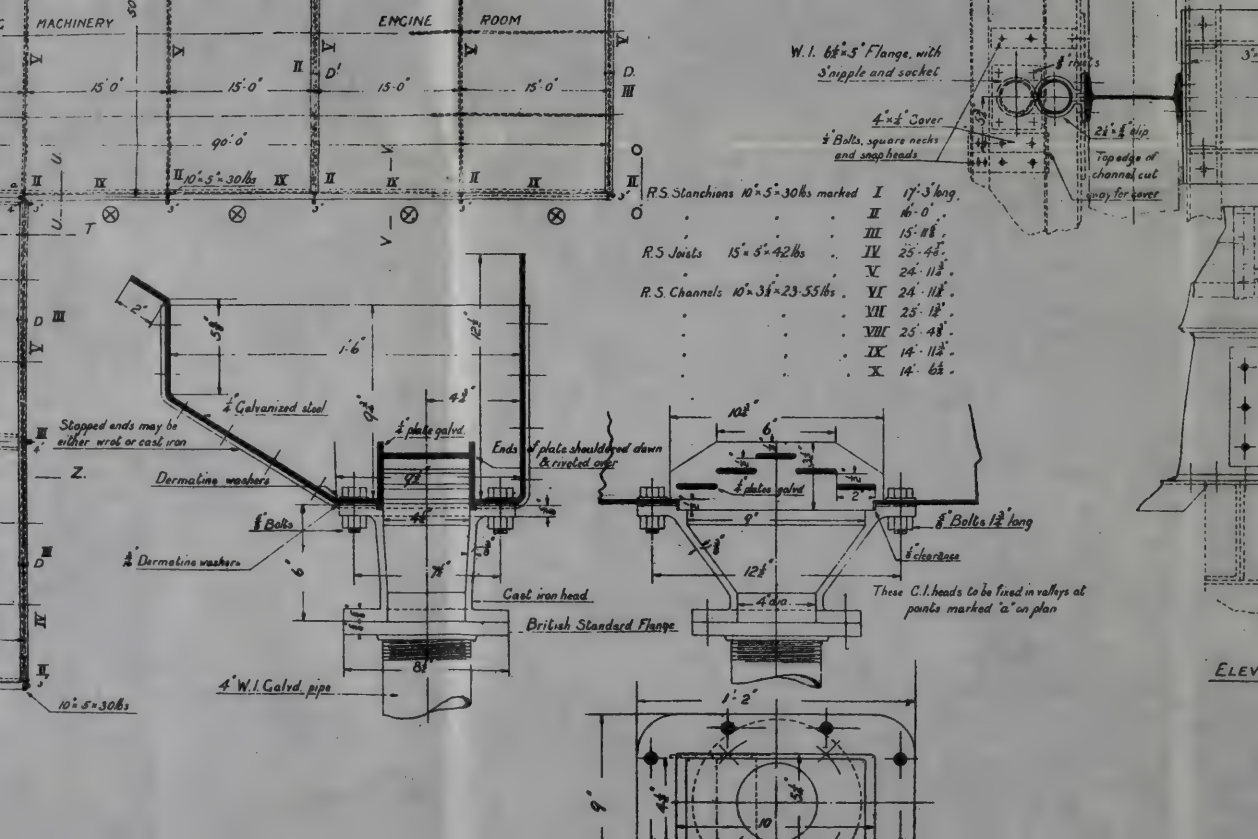
10 x 3½ x 23.55 lbs channel

¾" bolts

10 x 5 x 30 lbs Steel Stanchion 17'-3" long



Positions of intermediate stanchions marked D. D.' D.² to be arranged to suit doorways


MACHINERY
ENGINE ROOM
15'0"
15'0"
15'0"
15'0"
90'0"
10"×5"×30lbs
W.I. 6½"×5" Flange, with 3 nipple and socket
4"×½" Cover
½" Bolts, square necks and snap heads
2½"×½" Clip
Top edge of channel cut away for cover
R.S. Stanchions 10"×5"×30lbs marked I 17'3" long.
II 16'0"
III 15'8"
R.S. Joists 15"×5"×42lbs IV 25'4"
V 24'11½"
R.S. Channels 10"×3½"×23·55lbs VI 24'11½"
VII 25'1"
VIII 25'4"
IX 14'11½"
X 14'6½"
Galvanized steel
Stopped ends may be either wrot or cast iron
Dermatine washers
Bolts
Dermatine washers
Cast iron head
British Standard Flange
4" W.I. Galvd. pipe
10"×5"×30lbs
½" plate galvd.
Ends of plate shouldered down & riveted over
½" plate galvd.
⅝" Bolts 1½" long
⅛" clearance
These C.I. heads to be fixed in valleys at points marked "a" on plan
ELEV

STEEL ROOFS

Plate III

5"×3" Purlins Joints to be made over principals, and to be scarfed Two ½" bolts with square washers to each joint

Corrugated steel roofing in two sheets 18 S.W.G. 3" patn with 9" end lap.

2 rivets in every corrugation and 6" apart at sides

Sheets to be fixed to purlins by galvanized screws 4" long ⅜" dia 9" apart with Dermatine and galvanized iron curved washers

3"×3"×⅜" Angle cleat

2-⅜" Plates

¼" Boarding nailed to wood purlins

Felt lining on top of boarding

⅜" Bolts with Dermatine and galvanized iron curved washers 12" centres

¼" Galvanized steel gutters in 8'-4" lengths, except end lengths, with 4"×¼" cover one end riveted, the other end bolted

5"×3"×⅜" Angle steel purlins

3"×3"×⅜" Angle cleat

3"×3"×⅜" Tee

3"×3"×⅜" Tee

⅜" Bolt & washer

Two ⅜" Rivets

Two ⅜" plates

⅜" Bolts

3"×⅜" Bar

12"

3"×⅜" Bars

3"×⅜" Bar

½" Bolts

Two ⅜" plates

¾" Bolts 2½" long

5"×3½"×½" Tee

⅜" Bolts (with square tapered washers) 2½" long

1'-5½"

15"×5"×42 lbs

⅜" Bolts & washers

10"×3½"×23·55 lbs channel

⅜" Plate

1'-6"

14'-6"

⅜" plate Edge of plate in front of stanchion

3"×3"×⅜" Angle

¾" Rivets

¾" Bolt

⅜" Bolts, holes to be drilled in stanchions and plates but not in channels

SECTION B.B.

⅜" Plate 10"×5"

10"×5"×30 lbs stanchion 17'-3" long

Details of top of stanchions D

SECTION C.C.

¾" Bolts

3½"×23·55 lbs channel

2"×2"×⅜" Angle

7" Square flange 3"×3"×⅜" Angle

5"×3" Reducing Sockets

3" W.I. Galvd pipe

¼" plate

¼" rivets

ELEVATION N.N.

5"×3" Purlins

3"×3"×⅜" Tee

Two ⅝" Bolts on end trusses

⅝" Bolts square necks, snap heads

12"×8"×¼" Gutter with joint covers, stopped ends, & outlets for 3" & 4" pipes

12"

2½"×2½"×⅜" Angle

5"×3"×⅜" Angle bolted to top of 15"×5"×42 lbs joist with ⅝" bolts 12" pitch

Two ½" plates

¾" Bolts

12"

Bolts ¾" dia 2½" long

¾"×5"×1'-5½" plate

¾" Plates & ¾" Bolt

¾" rivets

⅜" angle plate

8"

¾" Bolts 2¾" long

¾" bolts

¾" bolts 2½" long with sq. tapered washers

⅜" Plate

10"×5"×30 lbs 16'-0" long

SECTION E.E.

SECTION F.F.

4" W.I. Galvd Pipe

7"×3½"×½" Angles

½" Plate

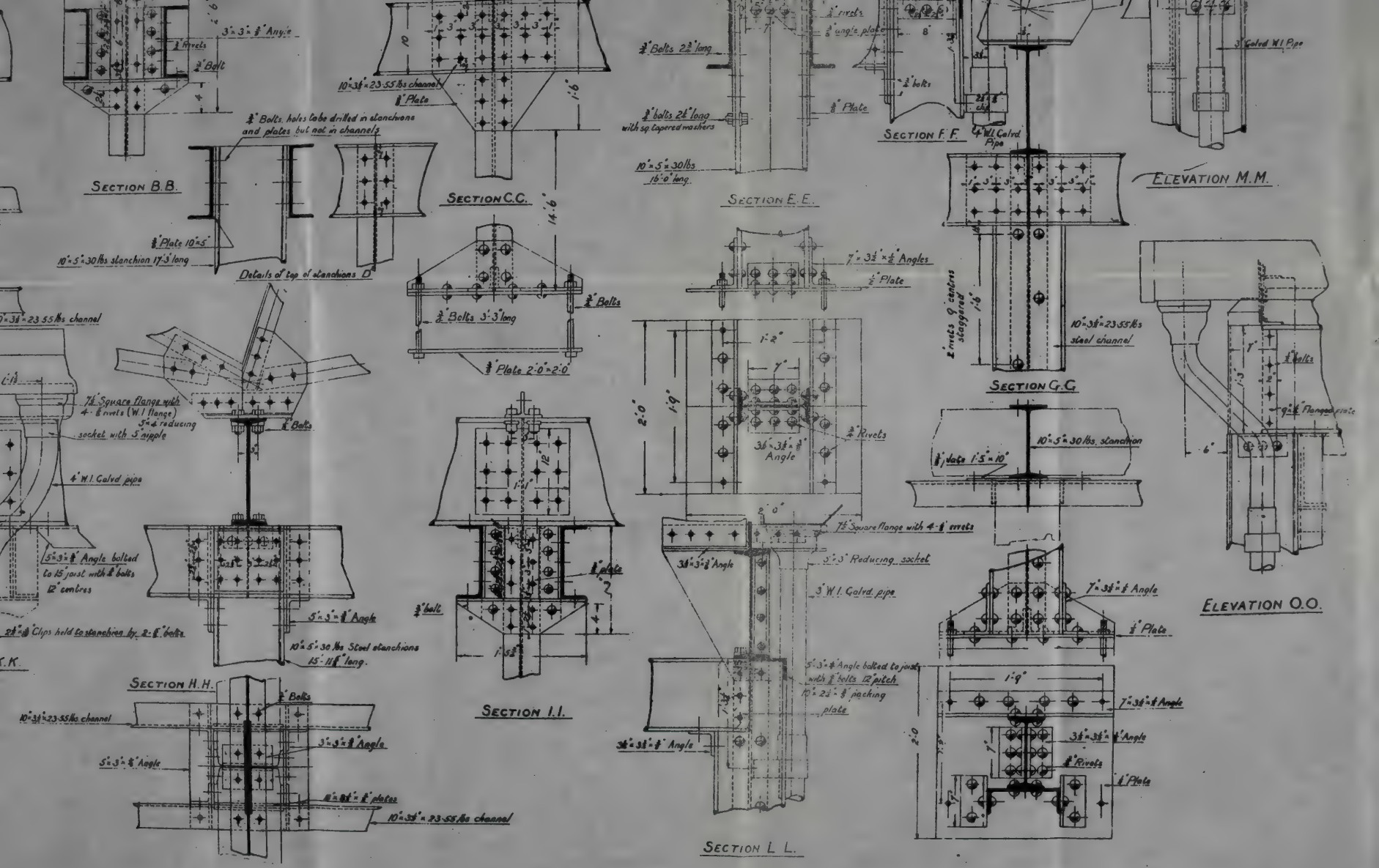
¾" bolts

2"×2"×⅜" angle

⅜" Plate

3" Galvd W.I. Pipe

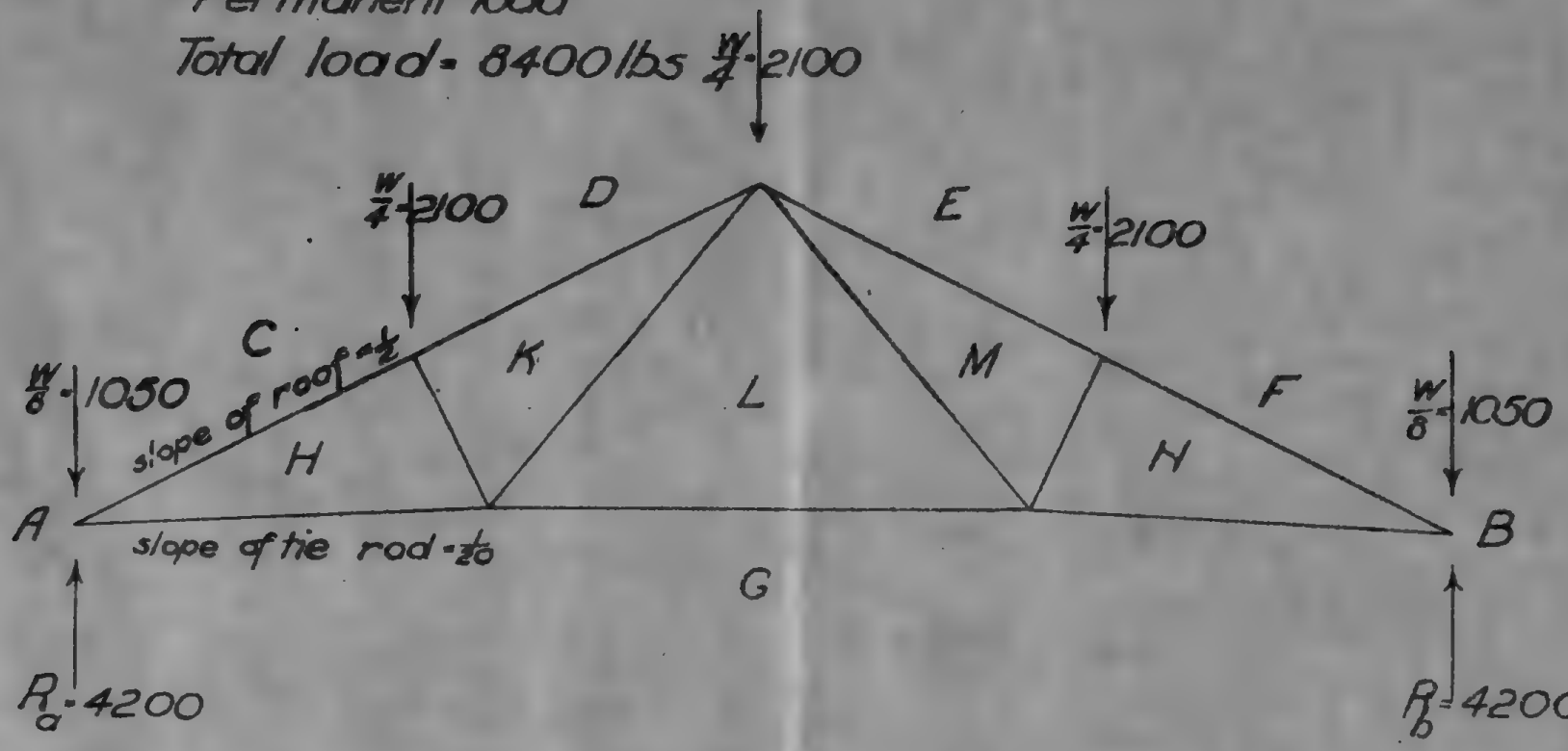
ELEVATION M.M.


3"×3"×⅜" Angle
⅜" Rivets
⅝" Bolt
10"×35#=23.55lbs channels
⅜" Plate
⅝" Bolts, holes to be drilled in stanchions and plates but not in channels
SECTION B.B.
⅜" Plate 10"×5"
10"×5"×30lbs stanchion 17'3" long
Details of top of stanchions D
SECTION C.C.
⅝" Bolts 3'-3" long
⅜" Plate 2'0"×2'0"
⅝" Bolts
14'6"
⅝" Bolts 22" long
⅝" bolts 24" long with sq tapered washers
10"×5"×30lbs 16'0" long
⅜" Plate
SECTION E.E.
2" rivets
⅜" angle plate
⅝" bolts
4" W.I. Galvd Pipe
SECTION F.F.
3" Galvd W.I. Pipe
ELEVATION M.M.
7"×3½"×½" Angles
½" Plate
2" rivets 9" centres staggered
10"×35#=23.55lbs steel channel
SECTION G.G.
10"×35#=23.55lbs channel
7½" Square flange with 4-⅝" rivets (W.I. flange)
5"×4" reducing socket with 5" nipple
⅝" Bolts
4" W.I. Galvd pipe
5"×3"×⅜" Angle bolted to 15" joist with ⅝" bolts 12" centres
2½"×⅜" Clips held to stanchion by 2-⅝" bolts
K.K.
5"×3"×⅜" Angle
10"×5"×30 lbs Steel stanchions 15'11" long.
SECTION H.H.
⅝" Bolts
10"×35#=23.55lbs channel
3"×3"×⅜" Angle
5"×3"×⅜" Angle
10"×35#=23.55lbs channel
SECTION I.I.
⅝" plate
⅝" bolt
1'-5½"
2'-0"
1'-9"
36"×3½"×⅜" Angle
⅝" Rivets
7½" Square flange with 4-⅝" rivets
5"×3" Reducing socket
3" W.I. Galvd pipe
3½"×3"×⅜" Angle
5"×3"×⅜" Angle bolted to joist with ⅝" bolts 12" pitch
10"×25"×⅜" packing plate
36"×3½"×⅜" Angle
SECTION L.L.
10"×5"×30 lbs. stanchion
⅜" plate 1'5"×10"
7"×3½"×⅜" Angle
⅜" Plate
1'-9"
7"×3½"×⅜" Angle
3½"×3½"×⅜" Angle
⅝" Rivets
1" Plate
2'0"
ELEVATION O.O.
⅝" bolts
9"×⅜" Flanged plate

SS DI

Fig 16a

Permanent load
Total load - 8400 lbs



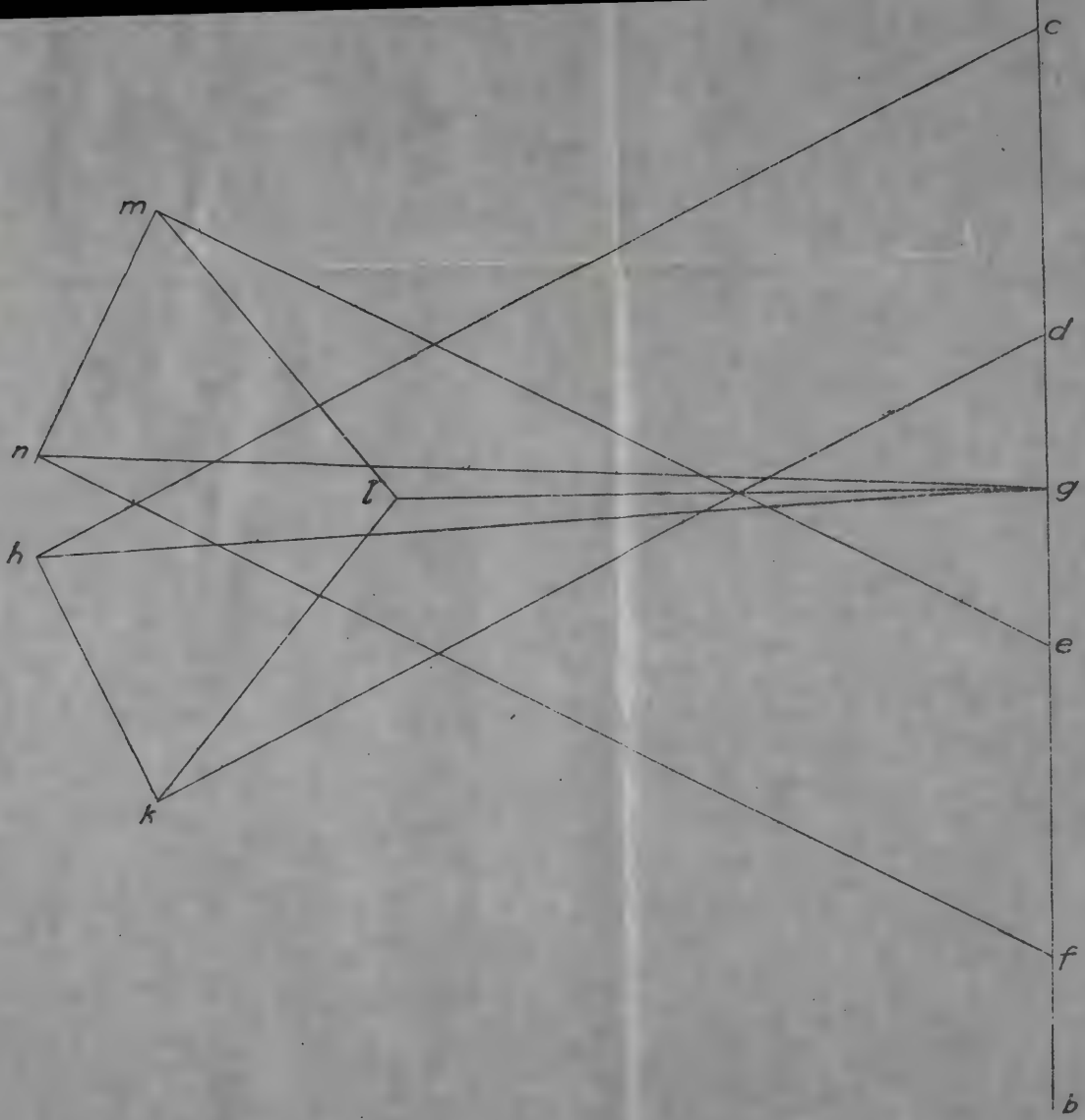
Stress diagram

TRUSSES

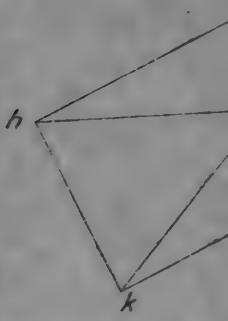
Fig 16b

Wind on left, ends fixed
Total wind - 2520 lbs

Stress Diagram



Stress Diagram



1000 500

LOAD AND STRESS DIAGRAMS FOR A STEEL ROOF

TRUSSES 24 FT CLEAR SPAN. AT 7FT 6IN C TO C. TO CARRY DOUBLE

Fig 16b

on left, ends fixed.
wind - 2520 lbs

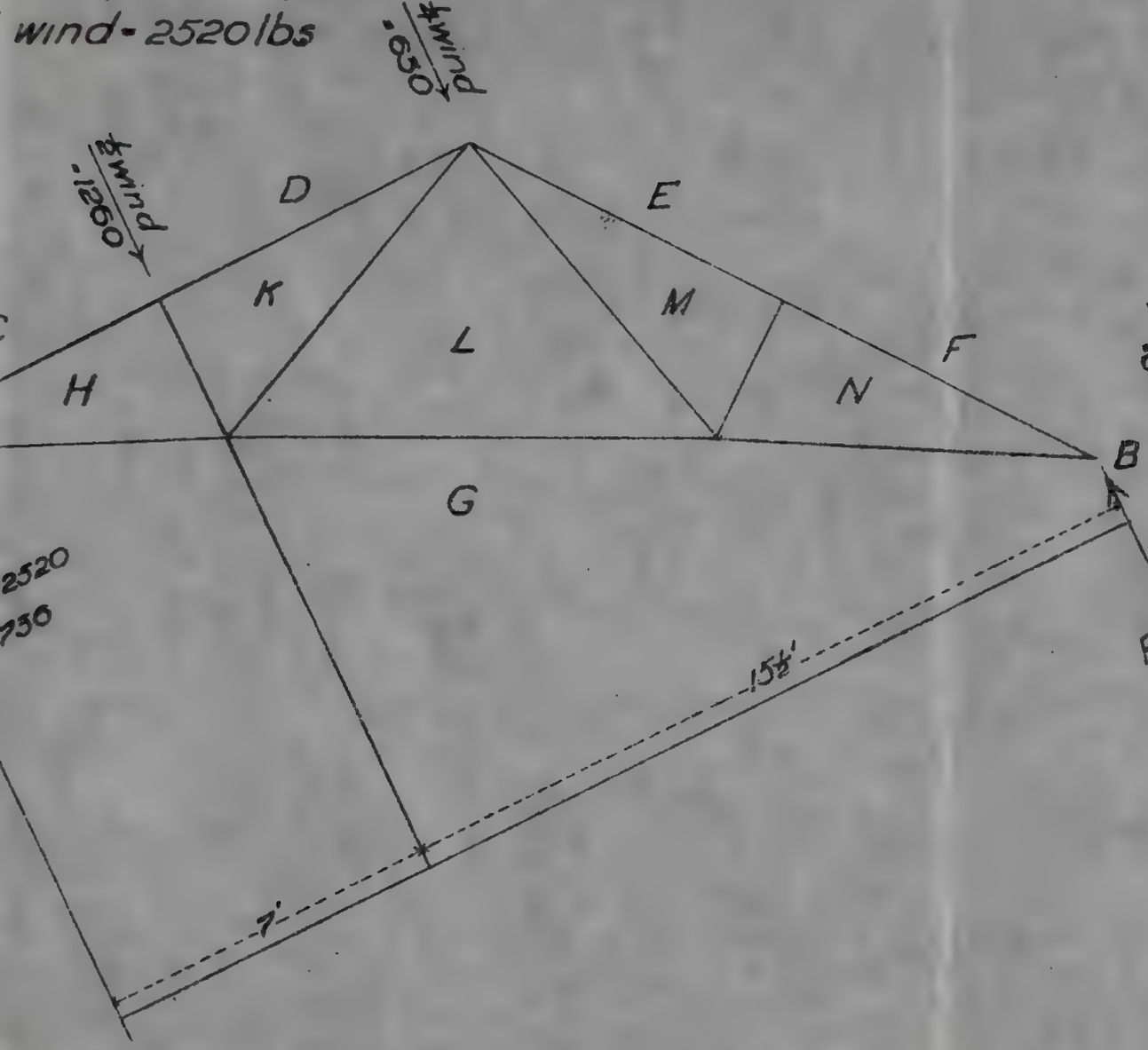
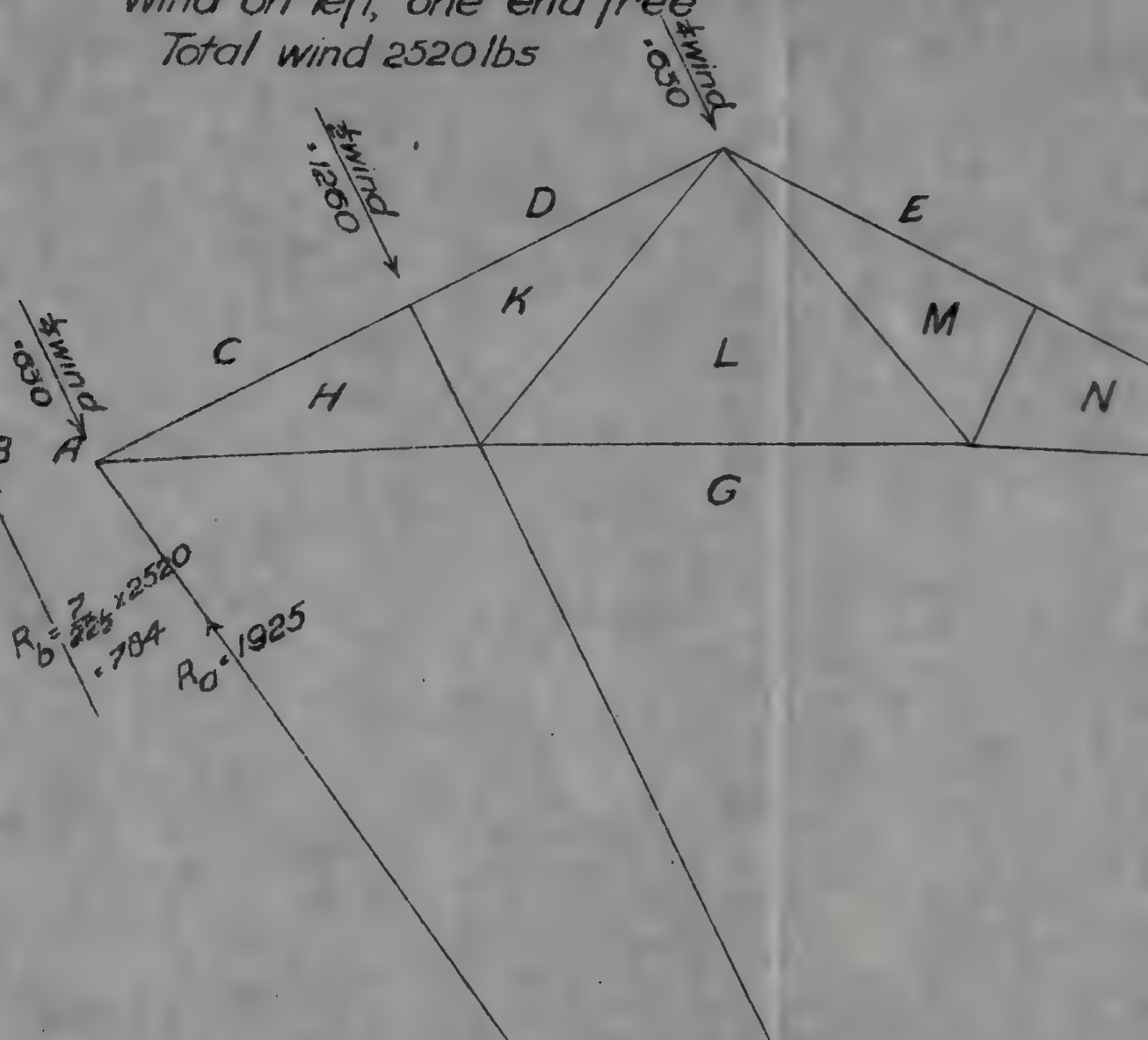
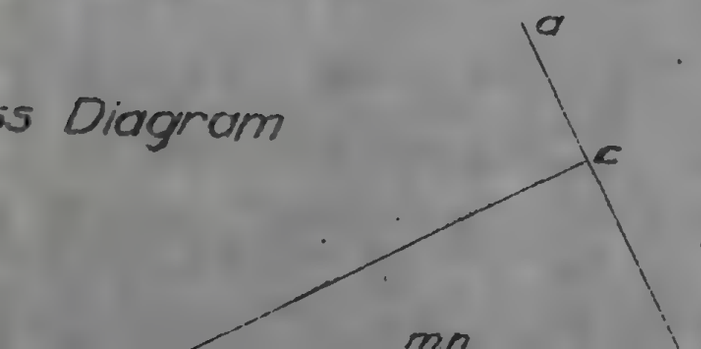


Fig 16c

Wind on left, one end free
Total wind 2520 lbs



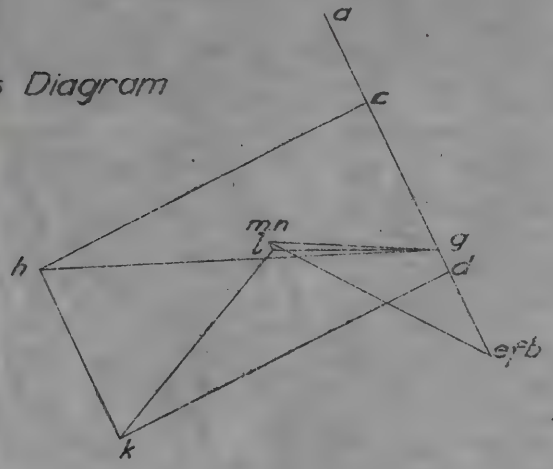
ss Diagram



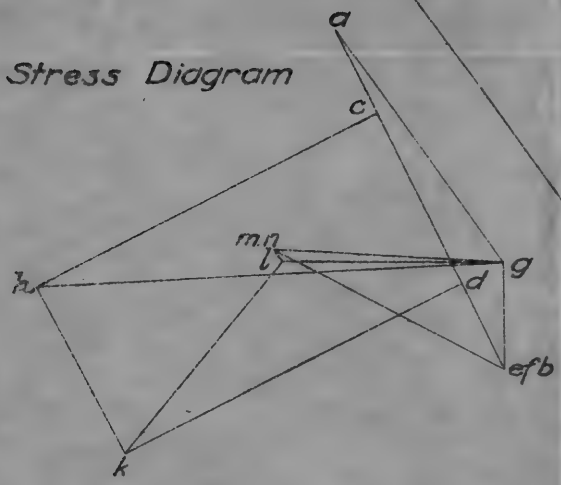
Stress Diagram

7'

ess Diagram



Stress Diagram



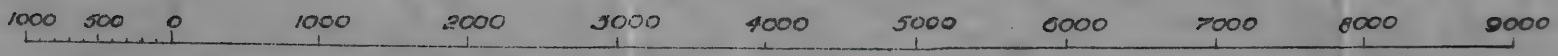
700

1925

2520

SCALE of FORCES

1000 lbs. = 1 inch



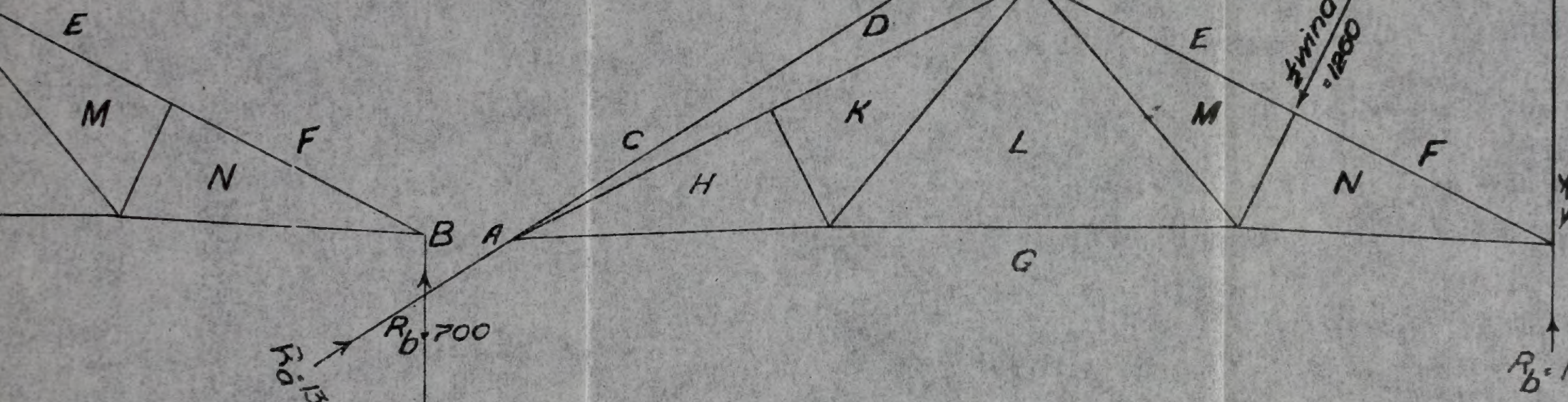
EL ROOF TRUSS.

STEEL ROOFS *Plate IV*

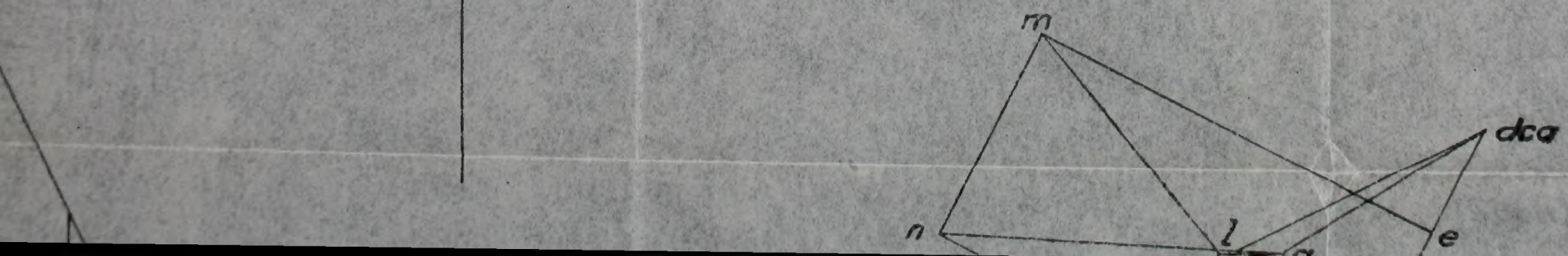
RY DOUBLE ALLAHABAD TILES.

Fig 16d

Wind on right, one end free
Total wind - 2520 lbs.



Stress diagram



Stress diagram